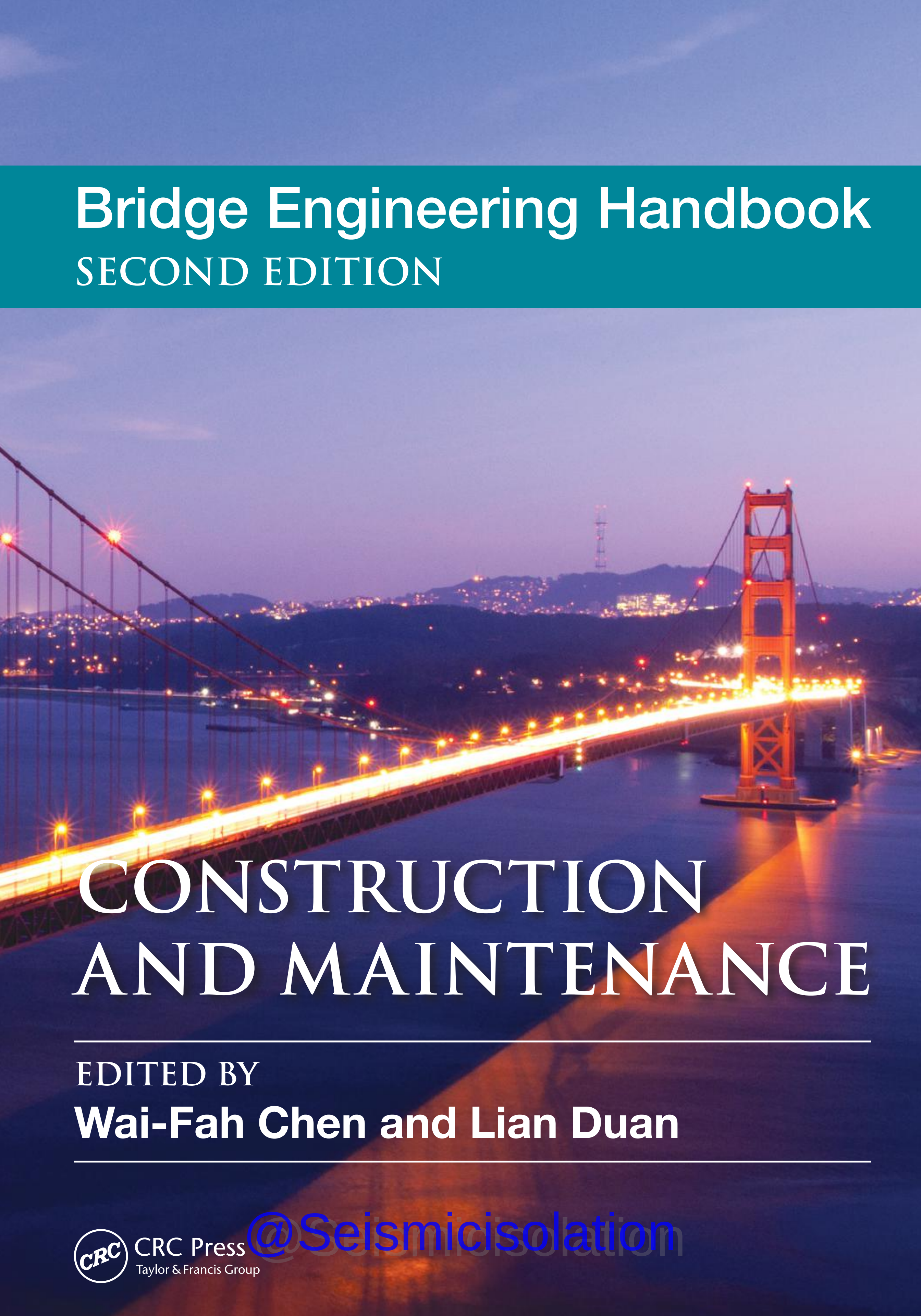
Bridge Engineering Handbook

SECOND EDITION



CONSTRUCTION AND MAINTENANCE

EDITED BY

Wai-Fah Chen and Lian Duan

CRC Press
Taylor & Francis Group

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Bridge Engineering Handbook, Second Edition

Bridge Engineering Handbook, Second Edition: Fundamentals

Bridge Engineering Handbook, Second Edition: Superstructure Design

Bridge Engineering Handbook, Second Edition: Substructure Design

Bridge Engineering Handbook, Second Edition: Seismic Design

Bridge Engineering Handbook, Second Edition: Construction and Maintenance

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CRC Press
Taylor & Francis Group
Boca Raton London New York

CRC Press is an imprint of the
Taylor & Francis Group, an **informa** business

CRC Press
Taylor & Francis Group
6000 Broken Sound Parkway NW, Suite 300
Boca Raton, FL 33487-2742

Version Date: 20130923

International Standard Book Number-13: 978-1-4398-5233-0 (eBook - PDF)

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Foreword

Throughout the history of civilization bridges have been the icons of cities, regions, and countries. All bridges are useful for transportation, commerce, and war. Bridges are necessary for civilization to exist, and many bridges are beautiful. A few have become the symbols of the best, noblest, and most beautiful that mankind has achieved. The secrets of the design and construction of the ancient bridges have been lost, but how could one not marvel at the magnificence, for example, of the Roman viaducts?

The second edition of the *Bridge Engineering Handbook* expands and updates the previous edition by including the new developments of the first decade of the twenty-first century. Modern bridge engineering has its roots in the nineteenth century, when wrought iron, steel, and reinforced concrete began to compete with timber, stone, and brick bridges. By the beginning of World War II, the transportation infrastructure of Europe and North America was essentially complete, and it served to sustain civilization as we know it. The iconic bridge symbols of modern cities were in place: Golden Gate Bridge of San Francisco, Brooklyn Bridge, London Bridge, Eads Bridge of St. Louis, and the bridges of Paris, Lisbon, and the bridges on the Rhine and the Danube. Budapest, my birthplace, had seven beautiful bridges across the Danube. Bridge engineering had reached its golden age, and what more and better could be attained than that which was already achieved?

Then came World War II, and most bridges on the European continent were destroyed. All seven bridges of Budapest were blown apart by January 1945. Bridge engineers after the war were suddenly forced to start to rebuild with scant resources and with open minds. A renaissance of bridge engineering started in Europe, then spreading to America, Japan, China, and advancing to who knows where in the world, maybe Siberia, Africa? It just keeps going! The past 60 years of bridge engineering have brought us many new forms of bridge architecture (plate girder bridges, cable stayed bridges, segmental prestressed concrete bridges, composite bridges), and longer spans. Meanwhile enormous knowledge and experience have been amassed by the profession, and progress has benefitted greatly by the availability of the digital computer. The purpose of the *Bridge Engineering Handbook* is to bring much of this knowledge and experience to the bridge engineering community of the world. The contents encompass the whole spectrum of the life cycle of the bridge, from conception to demolition.

The editors have convinced 146 experts from many parts of the world to contribute their knowledge and to share the secrets of their successful and unsuccessful experiences. Despite all that is known, there are still failures: engineers are human, they make errors; nature is capricious, it brings unexpected surprises! But bridge engineers learn from failures, and even errors help to foster progress.

The *Bridge Engineering Handbook*, second edition consists of five books:

Fundamentals
Superstructure Design
Substructure Design
Seismic Design
Construction and Maintenance

Fundamentals, Superstructure Design, and *Substructure Design* present the many topics necessary for planning and designing modern bridges of all types, made of many kinds of materials and systems, and subject to the typical loads and environmental effects. *Seismic Design* and *Construction and Maintenance* recognize the importance that bridges in parts of the world where there is a chance of earthquake occurrences must survive such an event, and that they need inspection, maintenance, and possible repair throughout their intended life span. Seismic events require that a bridge sustain repeated dynamic load cycles without functional failure because it must be part of the postearthquake lifeline for the affected area. *Construction and Maintenance* touches on the many very important aspects of bridge management that become more and more important as the world's bridge inventory ages.

The editors of the *Bridge Engineering Handbook*, Second Edition are to be highly commended for undertaking this effort for the benefit of the world's bridge engineers. The enduring result will be a safer and more cost effective family of bridges and bridge systems. I thank them for their effort, and I also thank the 146 contributors.

Theodore V. Galambos, PE
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Preface to the Second Edition

In the approximately 13 years since the original edition of the *Bridge Engineering Handbook* was published in 2000, we have received numerous letters, e-mails, and reviews from readers including educators and practitioners commenting on the handbook and suggesting how it could be improved. We have also built up a large file of ideas based on our own experiences. With the aid of all this information, we have completely revised and updated the handbook. In writing this Preface to the Second Edition, we assume readers have read the original Preface. Following its tradition, the second edition handbook stresses professional applications and practical solutions; describes the basic concepts and assumptions omitting the derivations of formulas and theories; emphasizes seismic design, rehabilitation, retrofit and maintenance; covers traditional and new, innovative practices; provides over 2500 tables, charts, and illustrations in ready-to-use format and an abundance of worked-out examples giving readers step-by-step design procedures. The most significant changes in this second edition are as follows:

- The handbook of 89 chapters is published in five books: *Fundamentals, Superstructure Design, Substructure Design, Seismic Design,* and *Construction and Maintenance*.
- *Fundamentals,* with 22 chapters, combines Section I, Fundamentals, and Section VI, Special Topics, of the original edition and covers the basic concepts, theory and special topics of bridge engineering. Seven new chapters are Finite Element Method, High-Speed Railway Bridges, Structural Performance Indicators for Bridges, Concrete Design, Steel Design, High Performance Steel, and Design and Damage Evaluation Methods for Reinforced Concrete Beams under Impact Loading. Three chapters including Conceptual Design, Bridge Aesthetics: Achieving Structural Art in Bridge Design, and Application of Fiber Reinforced Polymers in Bridges, are completely rewritten. Three special topic chapters, Weigh-In-Motion Measurement of Trucks on Bridges, Impact Effect of Moving Vehicles, and Active Control on Bridge Engineering, were deleted.
- *Superstructure Design,* with 19 chapters, provides information on how to design all types of bridges. Two new chapters are Extradosed Bridges and Stress Ribbon Pedestrian Bridges. The Prestressed Concrete Girder Bridges chapter is completely rewritten into two chapters: Precast–Pretensioned Concrete Girder Bridges and Cast-In-Place Posttensioned Prestressed Concrete Girder Bridges. The Bridge Decks and Approach Slabs chapter is completely rewritten into two chapters: Concrete Decks and Approach Slabs. Seven chapters, including Segmental Concrete Bridges, Composite Steel I-Girder Bridges, Composite Steel Box Girder Bridges, Arch Bridges, Cable-Stayed Bridges, Orthotropic Steel Decks, and Railings, are completely rewritten. The chapter Reinforced Concrete Girder Bridges was deleted because it is rarely used in modern time.
- *Substructure Design* has 11 chapters and addresses the various substructure components. A new chapter, Landslide Risk Assessment and Mitigation, is added. The Geotechnical Consideration chapter is completely rewritten and retitled as Ground Investigation. The Abutments and

Retaining Structures chapter is divided in two and updated as two chapters: Abutments and Earth Retaining Structures.

- *Seismic Design*, with 18 chapters, presents the latest in seismic bridge analysis and design. New chapters include Seismic Random Response Analysis, Displacement-Based Seismic Design of Bridges, Seismic Design of Thin-Walled Steel and CFT Piers, Seismic Design of Cable-Supported Bridges, and three chapters covering Seismic Design Practice in California, China, and Italy. Two chapters of Earthquake Damage to Bridges and Seismic Design of Concrete Bridges have been rewritten. Two chapters of Seismic Design Philosophies and Performance-Based Design Criteria, and Seismic Isolation and Supplemental Energy Dissipation, have also been completely rewritten and retitled as Seismic Bridge Design Specifications for the United States, and Seismic Isolation Design for Bridges, respectively. Two chapters covering Seismic Retrofit Practice and Seismic Retrofit Technology are combined into one chapter called Seismic Retrofit Technology.
- *Construction and Maintenance* has 19 chapters and focuses on the practical issues of bridge structures. Nine new chapters are Steel Bridge Fabrication, Cable-Supported Bridge Construction, Accelerated Bridge Construction, Bridge Management Using Pontis and Improved Concepts, Bridge Maintenance, Bridge Health Monitoring, Nondestructive Evaluation Methods for Bridge Elements, Life-Cycle Performance Analysis and Optimization, and Bridge Construction Methods. The Strengthening and Rehabilitation chapter is completely rewritten as two chapters: Rehabilitation and Strengthening of Highway Bridge Superstructures, and Rehabilitation and Strengthening of Orthotropic Steel Bridge Decks. The Maintenance Inspection and Rating chapter is completely rewritten as three chapters: Bridge Inspection, Steel Bridge Evaluation and Rating, and Concrete Bridge Evaluation and Rating.
- The section on Worldwide Practice in the original edition has been deleted, including the chapters on Design Practice in China, Europe, Japan, Russia, and the United States. An international team of bridge experts from 26 countries and areas in Africa, Asia, Europe, North America, and South America, has joined forces to produce the *Handbook of International Bridge Engineering, Second Edition*, the first comprehensive, and up-to-date resource book covering the state-of-the-practice in bridge engineering around the world. Each of the 26 country chapters presents that country's historical sketch; design specifications; and various types of bridges including girder, truss, arch, cable-stayed, suspension, and so on, in various types of materials—stone, timber, concrete, steel, advanced composite, and of varying purposes—highway, railway, and pedestrian. Ten benchmark highway composite girder designs, the highest bridges, the top 100 longest bridges, and the top 20 longest bridge spans for various bridge types are presented. More than 1650 beautiful bridge photos are provided to illustrate great achievements of engineering professions.

The 146 bridge experts contributing to these books have written chapters to cover the latest bridge engineering practices, as well as research and development from North America, Europe, and Pacific Rim countries. More than 80% of the contributors are practicing bridge engineers. In general, the handbook is aimed toward the needs of practicing engineers, but materials may be re-organized to accommodate several bridge courses at the undergraduate and graduate levels.

The authors acknowledge with thanks the comments, suggestions, and recommendations made during the development of the second edition of the handbook by Dr. Erik Yding Andersen, COWI A/S, Denmark; Michael J. Abrahams, Parsons Brinckerhoff, Inc.; Dr. Xiaohua Cheng, New Jersey Department of Transportation; Joyce E. Copelan, California Department of Transportation; Prof. Dan M. Frangopol, Lehigh University; Dr. John M. Kulicki, Modjeski and Masters; Dr. Amir M. Malek, California Department of Transportation; Teddy S. Theryo, Parsons Brinckerhoff, Inc.; Prof. Shouji Toma, Horrai-Gakuen University, Japan; Dr. Larry Wu, California Department of Transportation; Prof. Eiki Yamaguchi, Kyushu Institute of Technology, Japan; and Dr. Yi Edward Zhou, URS Corp.

We thank all the contributors for their contributions and also acknowledge Joseph Clements, acquiring editor; Jennifer Ahringer, project coordinator; and Joette Lynch, project editor, at Taylor & Francis/CRC Press.

Preface to the First Edition

The *Bridge Engineering Handbook* is a unique, comprehensive, and state-of-the-art reference work and resource book covering the major areas of bridge engineering with the theme "bridge to the twenty-first century." It has been written with practicing bridge and structural engineers in mind. The ideal readers will be MS-level structural and bridge engineers with a need for a single reference source to keep abreast of new developments and the state-of-the-practice, as well as to review standard practices.

The areas of bridge engineering include planning, analysis and design, construction, maintenance, and rehabilitation. To provide engineers a well-organized, user-friendly, and easy-to-follow resource, the handbook is divided into seven sections. Section I, Fundamentals, presents conceptual design, aesthetics, planning, design philosophies, bridge loads, structural analysis, and modeling. Section II, Superstructure Design, reviews how to design various bridges made of concrete, steel, steel-concrete composites, and timbers; horizontally curved, truss, arch, cable-stayed, suspension, floating, movable, and railroad bridges; and expansion joints, deck systems, and approach slabs. Section III, Substructure Design, addresses the various substructure components: bearings, piers and columns, towers, abutments and retaining structures, geotechnical considerations, footings, and foundations. Section IV, Seismic Design, provides earthquake geotechnical and damage considerations, seismic analysis and design, seismic isolation and energy dissipation, soil–structure–foundation interactions, and seismic retrofit technology and practice. Section V, Construction and Maintenance, includes construction of steel and concrete bridges, substructures of major overwater bridges, construction inspections, maintenance inspection and rating, strengthening, and rehabilitation. Section VI, Special Topics, addresses in-depth treatments of some important topics and their recent developments in bridge engineering. Section VII, Worldwide Practice, provides the global picture of bridge engineering history and practice from China, Europe, Japan, and Russia to the U.S.

The handbook stresses professional applications and practical solutions. Emphasis has been placed on ready-to-use materials, and special attention is given to rehabilitation, retrofit, and maintenance. The handbook contains many formulas and tables that give immediate answers to questions arising from practical works. It describes the basic concepts and assumptions, omitting the derivations of formulas and theories, and covers both traditional and new, innovative practices. An overview of the structure, organization, and contents of the book can be seen by examining the table of contents presented at the beginning, while the individual table of contents preceding each chapter provides an in-depth view of a particular subject. References at the end of each chapter can be consulted for more detailed studies.

Many internationally known authors have written the chapters from different countries covering bridge engineering practices, research, and development in North America, Europe, and the Pacific Rim. This handbook may provide a glimpse of a rapidly growing trend in global economy in recent years toward international outsourcing of practice and competition in all dimensions of engineering.

In general, the handbook is aimed toward the needs of practicing engineers, but materials may be reorganized to accommodate undergraduate and graduate level bridge courses. The book may also be used as a survey of the practice of bridge engineering around the world.

The authors acknowledge with thanks the comments, suggestions, and recommendations during the development of the handbook by Fritz Leonhardt, Professor Emeritus, Stuttgart University, Germany; Shouji Toma, Professor, Horrai-Gakuen University, Japan; Gerard F. Fox, Consulting Engineer; Jackson L. Durkee, Consulting Engineer; Michael J. Abrahams, Senior Vice President, Parsons, Brinckerhoff, Quade & Douglas, Inc.; Ben C. Gerwick, Jr., Professor Emeritus, University of California at Berkeley; Gregory F. Fenves, Professor, University of California at Berkeley; John M. Kulicki, President and Chief Engineer, Modjeski and Masters; James Chai, Senior Materials and Research Engineer, California Department of Transportation; Jinrong Wang, Senior Bridge Engineer, URS Greiner; and David W. Liu, Principal, Imbsen & Associates, Inc.

We thank all the authors for their contributions and also acknowledge at CRC Press Nora Konopka, acquiring editor, and Carol Whitehead and Sylvia Wood, project editors.

Editors

Dr. Wai-Fah Chen is a research professor of civil engineering at the University of Hawaii. He was dean of the College of Engineering at the University of Hawaii from 1999 to 2007, and a George E. Goodwin Distinguished Professor of Civil Engineering and head of the Department of Structural Engineering at Purdue University from 1976 to 1999.

He earned his BS in civil engineering from the National Cheng-Kung University, Taiwan, in 1959, MS in structural engineering from Lehigh University in 1963, and PhD in solid mechanics from Brown University in 1966. He received the Distinguished Alumnus Award from the National Cheng-Kung University in 1988 and the Distinguished Engineering Alumnus Medal from Brown University in 1999.

Dr. Chen's research interests cover several areas, including constitutive modeling of engineering materials, soil and concrete plasticity, structural connections, and structural stability. He is the recipient of several national engineering awards, including the Raymond Reese Research Prize and the Shortridge Hardesty Award, both from the American Society of Civil Engineers, and the T. R. Higgins Lectureship Award in 1985 and the Lifetime Achievement Award, both from the American Institute of Steel Construction. In 1995, he was elected to the U.S. National Academy of Engineering. In 1997, he was awarded Honorary Membership by the American Society of Civil Engineers, and in 1998, he was elected to the Academia Sinica (National Academy of Science) in Taiwan.

A widely respected author, Dr. Chen has authored and coauthored more than 20 engineering books and 500 technical papers. His books include several classical works such as *Limit Analysis and Soil Plasticity* (Elsevier, 1975), the two-volume *Theory of Beam-Columns* (McGraw-Hill, 1976 and 1977), *Plasticity in Reinforced Concrete* (McGraw-Hill, 1982), and the two-volume *Constitutive Equations for Engineering Materials* (Elsevier, 1994). He currently serves on the editorial boards of more than 15 technical journals.

Dr. Chen is the editor-in-chief for the popular *Civil Engineering Handbook* (CRC Press, 1995 and 2003), the *Handbook of Structural Engineering* (CRC Press, 1997 and 2005), the *Earthquake Engineering Handbook* (CRC Press, 2003), the *Semi-Rigid Connections Handbook* (J. Ross Publishing, 2011), and the *Handbook of International Bridge Engineering* (CRC Press, 2014). He currently serves as the consulting editor for the *McGraw-Hill Yearbook of Science & Technology* for the field of civil and architectural engineering.

He was a longtime member of the executive committee of the Structural Stability Research Council and the specification committee of the American Institute of Steel Construction. He was a consultant for Exxon Production Research on offshore structures, for Skidmore, Owings, and Merrill in Chicago on tall steel buildings, and for the World Bank on the Chinese University Development Projects, among many others. Dr. Chen has taught at Lehigh University, Purdue University, and the University of Hawaii.

Dr. Lian Duan is a senior bridge engineer and structural steel committee chair with the California Department of Transportation (Caltrans). He worked at the North China Power Design Institute from 1975 to 1978 and taught at Taiyuan University of Technology, China, from 1981 to 1985.

He earned his diploma in civil engineering in 1975, MS in structural engineering in 1981 from Taiyuan University of Technology, China, and PhD in structural engineering from Purdue University in 1990.

Dr. Duan's research interests cover areas including inelastic behavior of reinforced concrete and steel structures, structural stability, seismic bridge analysis, and design. With more than 70 authored and coauthored papers, chapters, and reports, his research focuses on the development of unified interaction equations for steel beam-columns, flexural stiffness of reinforced concrete members, effective length factors of compression members, and design of bridge structures.

Dr. Duan has over 35 years experience in structural and bridge engineering. He was lead engineer for the development of Caltrans *Guide Specifications for Seismic Design of Steel Bridges*. He is a registered professional engineer in California. He served as a member for several National Highway Cooperative Research Program panels and was a Transportation Research Board Steel Committee member from 2000 to 2006.

He is the coeditor of the *Handbook of International Bridge Engineering,* (CRC Press, 2014). He received the prestigious 2001 Arthur M. Wellington Prize from the American Society of Civil Engineers for the paper, "Section Properties for Latticed Members of San Francisco-Oakland Bay Bridge," in the *Journal of Bridge Engineering,* May 2000. He received the Professional Achievement Award from Professional Engineers in California Government in 2007 and the Distinguished Engineering Achievement Award from the Engineers' Council in 2010.

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1

Steel Bridge Construction

Jackson Durkee
Consulting Structural Engineer

1.1 Introduction

This chapter addresses some of the principles and practices applicable to the construction of medium- and long-span steel bridges—structures of such size and complexity that construction engineering becomes an important or even the governing factor in the successful fabrication and erection of the superstructure steelwork.

We begin with an explanation of the fundamental nature of construction engineering, then go on to explain some of the challenges and obstacles involved. The basic considerations of cambering are explained. Two general approaches to the fabrication and erection of bridge steelwork are described, with examples from experience with arch bridges, suspension bridges, and cable-stayed bridges.

The problem of erection-strength adequacy of trusswork under erection is considered, and a method of appraisal offered that is believed to be superior to the standard working-stress procedure.

Typical problems with respect to construction procedure drawings, specifications, and practices are reviewed, and methods for improvement are suggested. The need for comprehensive bridge erection-engineering specifications, and for standard conditions for contracting, is set forth, and the design-and-construct contracting procedure is described.

Finally, we take a view ahead, to the future prospects for effective construction engineering in the United States.

The chapter also contains a large number of illustrations showing a variety of erection methods for several types of major steel bridges.

1.2 Construction Engineering in Relation to Design Engineering

With respect to bridge steelwork, the differences between construction engineering and design engineering should be kept firmly in mind. Design engineering is of course a concept and process well known to structural engineers; it involves preparing a set of plans and specifications—known as the contract documents— that define the structure in its completed configuration, referred to as the geometric outline. Thus, the design drawings describe to the contractor the steel bridge superstructure that the owner wants to see in place when the project is completed. A considerable design engineering effort is required to prepare a good set of contract documents.

Construction engineering, however, is not so well known. It involves governing and guiding the fabrication and erection operations needed to produce the structural steel members to the proper cambered or "no-load" shape, and get them safely and efficiently "up in the air" in place in the structure, so that the completed structure under the dead-load conditions and at normal temperature will meet the geometric and stress requirements stipulated on the design drawings.

Four key considerations may be noted: (1) design engineering is widely practiced and reasonably well understood, and is the subject of a steady stream of technical papers; (2) construction engineering is practiced on only a limited basis, is not as well understood, and is hardly ever discussed; (3) for medium- and long-span bridges, the construction engineering aspects are likely to be no less important than design engineering aspects; and (4) adequately staffed and experienced construction engineering offices are a rarity.

1.3 Construction Engineering Can Be Critical

The construction phase of the total life of a major steel bridge will probably be much more hazardous than the service-use phase. Experience shows that a large bridge is more likely to suffer failure during erection than after completion. Many decades ago, steel bridge design engineering had progressed to the stage where the chance of structural failure under service loadings became altogether remote. However, the erection phase for a large bridge is inherently less secure, primarily because of the prospect of inadequacies

FIGURE 1.1 Failure of a steel girder bridge during erection, 1995. Steel bridge failures such as this one invite suspicion that the construction engineering aspects were not properly attended to.

in construction engineering and its implementation at the job site. The hazards associated with the erection of large steel bridges will be readily apparent from a review of the illustrations in this chapter.

For significant steel bridges, the key to construction integrity lies in the proper planning and engineering of steelwork fabrication and erection. Conversely, failure to attend properly to construction engineering constitutes an invitation to disaster. In fact, this thesis is so compelling that whenever a steel bridge failure occurs during construction (see, for example, Figure 1.1), it is reasonable to assume that the construction engineering investigation was inadequate, not properly implemented, or both.

1.4 Premises and Objectives of Construction Engineering

During the erection sequences, the various components of steel bridges may be subjected to stresses that are quite different from those that will occur under the service loadings and that have been provided for by the designer. For example, during construction there may be a derrick moving and working on the partially erected structure, and the structure may be cantilevered out some distance causing tension-designed members to be in compression and vice versa. Thus, the steelwork contractor needs to engineer the bridge members through their various construction loadings, and strengthen and stabilize them as may be necessary. Further, the contractor may need to provide temporary members to support and stabilize the structure as it passes through its successive erection configurations.

In addition to strength problems there are also geometric considerations. The steelwork contractor must engineer the construction sequences step by step to ensure that the structure will fit properly together as erection progresses, and that the final or closing members can be moved into position and connected. Finally, of course, the steelwork contractor must carry out the engineering studies needed to ensure that the geometry and stressing of the completed structure under normal temperature will be in accordance with the requirements of the design plans and specifications.

1.5 Fabrication and Erection Information Shown on Design Plans

Regrettably, the level of engineering effort required to accomplish safe and efficient fabrication and erection of steelwork superstructures is not widely understood or appreciated in bridge design offices, nor indeed by many steelwork contractors. It is only infrequently that we find a proper level of capability and effort in the engineering of construction.

The design drawings for an important bridge will sometimes display an erection scheme, even though most designers are not experienced in the practice of erection engineering and usually expend only a minimum or even superficial effort on erection studies. The scheme portrayed may not be practical, or may not be suitable in respect to the bidder or contractor's equipment and experience. Accordingly, the bidder or contractor may be making a serious mistake if he relies on an erection scheme portrayed on the design plans.

As an example of misplaced erection effort on the part of the designer, there have been cases where the design plans show cantilever erection by deck travelers, with the permanent members strengthened correspondingly to accommodate the erection loadings; but the successful bidder elected to use waterborne erection derricks with long booms, thereby obviating the necessity for most or all of the erection strengthening provided on the design plans. Further, even in those cases where the contractor would decide to erect by cantilevering as anticipated on the plans, there is hardly any way for the design engineer to know what will be the weight and dimensions of the contractor's erection travelers.

1.6 Erection Feasibility

Of course, the bridge designer does have a certain responsibility to his client and to the public in respect to the erection of the bridge steelwork. This responsibility includes (1) making certain, during the design stage, that there is a feasible and economical method to erect the steelwork; (2) setting forth in the contract documents any necessary erection guidelines and restrictions; and (3) reviewing the contractor's erection scheme, including any strengthening that may be needed, to verify its suitability. It may be noted that this latter review does not relieve the contractor from responsibility for the adequacy and safety of the field operations.

Bridge annals include a number of cases where the design engineer failed to consider erection feasibility. In one notable instance, the design plans showed the 1200 ft. (366 m) main span for a long crossing over a wide river as an aesthetically pleasing steel tied-arch. However, erection of such a span in the middle of the river was impractical; one bidder found that the tonnage of falsework required was about the same as the weight of the permanent arch-span steelwork. Following opening of the bids, the owner found the prices quoted to be well beyond the resources available, and the tied-arch main span was discarded in favor of a through-cantilever structure, for which erection falsework needs were minimal and practical.

It may be noted that design engineers can stand clear of serious mistakes such as this one, by the simple expedient of conferring with prospective bidders during the preliminary design stage of a major bridge.

1.7 Illustrations of Challenges in Construction Engineering

Space does not permit comprehensive coverage of the numerous and difficult technical challenges that can confront the construction engineer in the course of the erection of various types of major steel bridges. However, some conception of the kinds of steelwork erection problems, the methods available to resolve them, and the hazards involved can be conveyed by views of bridges in various stages of erection; refer to the illustrations in the text.

1.8 Obstacles to Effective Construction Engineering

There is an unfortunate tendency among design engineers to view construction engineering as relatively unimportant. This view may be augmented by the fact that few designers have had any significant experience in the engineering of construction.

Further, managers in the construction industry must look critically at costs, and they can readily develop the attitude that their engineers are doing unnecessary theoretical studies and calculations, detached from the practical world. (And indeed, this may sometimes be the case.) Such management apprehension can constitute a serious obstacle to staff engineers who see the need to have enough money in the bridge tender to cover a proper construction engineering effort for the project. There is the tendency for steelwork construction company management to cut back the construction engineering allowance, partly because of this apprehension and partly because of the concern that other tenderers will not be allotting adequate money for construction engineering. This effort is often thought of by company management as "a necessary evil" at best—something they would prefer not to be bothered with or burdened with.

Accordingly, construction engineering tends to be a difficult area of endeavor. The way for staff engineers to gain the confidence of management is obvious—they need to conduct their investigations to a level of technical proficiency that will command management respect and support, and they must keep management informed as to what they are doing and why it is necessary. As for management's concern that other bridge tenderers will not be putting into their packages much money for construction engineering, this concern is no doubt often justified, and it is difficult to see how responsible steelwork contractors can cope with this problem.

1.9 Examples of Inadequate Construction Engineering Allowances and Effort

Even with the best of intentions, the bidder's allocation of money to construction engineering can be inadequate. A case in point involved a very heavy, long-span cantilever truss bridge crossing a major river. The bridge superstructure carried a contract price of some \$30 million, including an allowance of \$150,000, or about one-half of 1%, for construction engineering of the permanent steelwork (i.e., not including such matters as design of erection equipment). As fabrication and erection progressed, many unanticipated technical problems came forward, including brittle-fracture aspects of certain grades of the high-strength structural steel, and aerodynamic instability of H-shaped vertical and diagonal truss members. In the end the contractor's construction engineering effort mounted to about \$1.3 million, almost nine times the estimated cost.

Another significant example—this one in the domain of buildings—involved a design-and-construct project for airplane maintenance hangars at a prominent international airport. There were two large and complicated buildings, each 100 × 150 m (328 × 492 ft.) in plan and 37 m (121 ft.) high with a 10 m (33 ft.) deep space-frame roof. Each building contained about 2450 tons of structural steelwork. The design-and-construct steelwork contractor had submitted a bid of about \$30 million, and included therein was the magnificent sum of \$5000 for construction engineering, under the expectation that this work could be done on an incidental basis by the project engineer in his "spare time."

As the steelwork contract went forward it quickly became obvious that the construction engineering effort had been grossly underestimated. The contractor proceeded to staff-up appropriately and carried out in-depth studies, leading to a detailed erection procedure manual of some 270 pages showing such matters as erection equipment and its positioning and clearances; falsework requirements; lifting tackle and jacking facilities; stress, stability, and geometric studies for gravity and wind loads; step-by-step instructions for raising, entering, and connecting the steelwork components; closing and swinging the roof structure and portal frame; and welding guidelines and procedures. This erection procedure manual turned out to be a key factor in the success of the fieldwork. The cost of this construction engineering effort amounted to about 10 times the estimate, but still came to a mere one-fifth of 1% of the total contract cost.

In yet another example, a major steelwork general contractor was induced to sublet the erection of a long-span cantilever truss bridge to a reputable erection contractor, whose quoted *price* for the work was less than the general contractor's estimated *cost*. During the erection cycle, the general contractor's engineers made some visits to the job site to observe progress, and were surprised and disconcerted to observe how little erection engineering and planning had been accomplished. For example, the erector had made no provision for installing jacks in the bottom-chord jacking points for closure of the main span; it was left up to the field forces to provide the jack bearing components inside the bottom-chord joints and to find the required jacks in the local market. When the job-built installations were tested it was discovered that they would not lift the cantilevered weight, and the job had to be shut down while the field engineer scouted around to find larger-capacity jacks. Further, certain compression members did not appear to be properly braced to carry the erection loadings; the erector had not engineered those members, but just assumed they were adequate. It became obvious that the erector had not appraised the bridge members for erection adequacy and had done little or no planning and engineering of the critical evolutions to be carried out in the field.

Many further examples of inadequate attention to construction engineering could be presented. Experience shows that the amounts of money and time allocated by steelwork contractors for the engineering of construction are frequently far less than desirable or necessary. Clearly, effort spent on construction engineering is worthwhile; it is obviously more efficient and cheaper, and certainly much safer, to plan and engineer steelwork construction in the office in advance of the work, rather than to leave these important matters for the field forces to work out. Just a few bad moves on site, with the corresponding waste of labor and equipment hours, will quickly use up sums of money much greater than those required for a proper construction engineering effort—not to mention the costs of any job accidents that might occur.

The obvious question is "Why is construction engineering not properly attended to?" Do not contractors learn, after a bad experience or two, that it is both necessary and cost effective to do a thorough job of planning and engineering the construction of important bridge projects? Experience and observation would seem to indicate that some steelwork contractors learn this lesson, while many do not. There is always pressure to reduce bid prices to the absolute minimum, and to add even a modest sum for construction engineering must inevitably reduce the prospect of being the low bidder.

1.10 Considerations Governing Construction Engineering Practices

There are no textbooks or manuals that define how to accomplish a proper job of construction engineering. In bridge construction (and no doubt in building construction as well) the engineering of construction tends to be a matter of each firm's experience, expertise, policies, and practices. Usually there is more than one way to build the structure, depending on the contractor's ingenuity and engineering skill, his risk appraisal and inclination to assume risk, the experience of his fabrication and erection work forces, his available equipment, and his personal preferences. Experience shows that each project is different; and although there will be similarities from one bridge of a given type to another, the construction engineering must be accomplished on an individual project basis. Many aspects of the project at hand will turn out to be different from those of previous similar jobs, and also there may be new engineering considerations and requirements for a given project that did not come forward on previous similar work.

During the estimating and bidding phase of the project the prudent, experienced bridge steelwork contractor will "start from scratch" and perform his own fabrication and erection studies, irrespective of any erection schemes and information that may be shown on the design plans. These studies can involve a considerable expenditure of both time and money, and thereby place that contractor at a disadvantage in respect to those bidders who are willing to rely on hasty, superficial studies, or—where the design engineer has shown an erection scheme—to simply assume that it has been engineered correctly and proceed to use it. The responsible contractor, on the other hand, will appraise the feasible construction methods and evaluate their costs and risks, and then make his selection.

After the contract has been executed, the contractor will set forth how he intends to fabricate and erect, in detailed plans that could involve a large number of calculation sheets and drawings along with construction procedure documents. It is appropriate for the design engineer on behalf of his client to review the contractor's plans carefully, perform a check of construction considerations, and raise appropriate questions. Where the contractor does not agree with the designer's comments the two parties get together for review and discussion, and in the end they concur on essential factors such as fabrication and erection procedures and sequences, the weight and positioning of erection equipment, the design of falsework and other temporary components, erection stressing and strengthening of the permanent steelwork, erection stability and bracing of critical components, any erection check measurements that may be needed, and span closing and swinging operations.

The design engineer's approval is needed for certain fabrication plans, such as the cambering of individual members; however, in most cases the designer should stand clear of actual *approval* of the contractor's construction plans since he is not in a position to accept construction responsibility, and too many things can happen during the field evolutions over which the designer has no control.

It should be emphasized that even though the design engineer usually has no significant experience in steelwork construction, the contractor should welcome his comments and evaluate them carefully and respectfully. In major bridge projects, many construction matters can be improved on or get out of control or can be improved upon, and the contractor should take advantage of every opportunity to improve his prospects and performance. The experienced contractor will make sure that he works constructively with the design engineer, standing well clear of antagonistic or confrontational posturing.

1.11 Camber Considerations

One of the first construction engineering problems to be resolved by the steel bridge contractor is the cambering of individual bridge components. The design plans will show the "geometric outline" of the bridge, which is its shape under the designated load condition—commonly full dead load—at normal temperature. The contractor, however, fabricates the bridge members under the no-load condition, and at the "shop temperature"—the temperature at which the shop measuring tapes have been standardized and will have the correct length. The difference between the shape of a member under full dead load and normal temperature, and its shape at the no-load condition and shop temperature, is defined as member camber.

While camber is inherently a simple concept, it is frequently misunderstood; indeed, it is often not correctly defined in design specifications and contract documents. For example, beam and girder camber has been defined in specifications as "the convexity induced into a member to provide for vertical curvature of grade and to offset the anticipated deflections indicated on the plans when the member is in its erected position in the structure. Cambers shall be measured in this erected position…" This definition is not correct and reflects a common misunderstanding of a key structural engineering term. Camber of bending members is not convexity, nor does it have anything to do with grade vertical curvature, nor is it measured with the member in the erected position. Camber—of a bending member, or any other member—is the *difference in shape* of the member under its no-load fabrication outline as compared with its geometric outline; and it is "measured"—the cambered dimensions are applied to the member—not when it is in the erected position (whatever that might be), but rather, when it is in the *no-load* condition.

In summary, camber is a *difference* in shape and not the shape itself. Beams and girders are commonly cambered to compensate for dead-load bending, and truss members to compensate for dead-load axial force. However, further refinements can be introduced as may be needed; for example, the arch-rib box members of the Lewiston–Queenston arch bridge (Figure 1.4, later in the chapter) were cambered to compensate for dead-load axial force, bending, and shear.

A further common misunderstanding regarding cambering of bridge members involves the effect of the erection scheme on cambers. The erection scheme may require certain members to be strengthened, and this in turn will affect the cambers of those members (and possibly of others as well, in the case of statically indeterminate structures). However, the fabricator should address the matter of cambering only after the final sizes of all bridge members have been determined. Camber is a function of member properties, and there is no merit to calculating camber for members whose cross-sectional areas may subsequently be increased because of erection forces.

Thus, the erection scheme may affect the required member properties, and these in turn will affect member cambering; but the erection scheme does not *of itself* have any effect on camber. Obviously, the temporary stress-and-strain maneuvers to which a member will be subjected, between its no-load condition in the shop and its full-dead-load condition in the completed structure, can have no bearing on the camber calculations for the member.

To illustrate the general principles that govern the cambering procedure, consider the main trusses of a truss bridge. The first step is to determine the erection procedure to be used, and to augment the strength of the truss members as may be necessary to sustain the erection forces. Next, the bridge dead-load weights are determined, and the member dead-load forces and effective cross-sectional areas are calculated.

Consider now a truss chord member having a geometric length of 49.1921 ft. panel-point to panel-point and an effective cross-sectional area of 344.5 in.^{2}, carrying a dead-load compressive force of 4230 kip. The bridge normal temperature is 45°F and the shop temperature is 68°F. We proceed as follows:

1. Assume that the chord member is in place in the bridge, at the full dead load of 4230 kip and the normal temperature of 45°F.
2. Remove the member from the bridge, allowing its compressive force to fall to zero. The member will increase in length by an amount ΔL_s:

$$\Delta L_s = \frac{SL}{AE} = \frac{4230 \text{ kip} \times 49.1921 \text{ ft.}}{344.5 \text{ in.}^2 \times 29{,}000 \text{ kip/in.}^2}$$
$$= 0.0208 \text{ ft.}$$

3. Now raise the member temperature from 45°F to 68°F. The member will increase in length by an additional amount ΔL_t:

$$\Delta L_t = L\omega t = (49.1921 + 0.0208) \text{ft.} \times 0.0000065/\text{deg} \times (68 - 45)\text{deg}$$
$$= 0.0074 \text{ ft.}$$

4. The total increase in member length will be

$$\Delta L = \Delta L_s + \Delta L_t = 0.0208 + 0.0074$$
$$= 0.0282 \text{ ft.}$$

5. The theoretical cambered member length—the no-load length at 68°F—will be

$$L_{tc} = 49.1921 + 0.0282 = 49.2203 \text{ ft.}$$

6. Rounding L_{tc} to the nearest 1/32 in., we obtain the cambered member length for fabrication as

$$L_{fc} = 49 \text{ ft. } 2\frac{21}{32} \text{ in.}$$

Accordingly, the general procedure for cambering a bridge member of any type can be summarized as follows:

1. Strengthen the structure to accommodate erection forces, as may be needed.
2. Determine the bridge dead-load weights, and the corresponding member dead-load forces and effective cross-sectional areas.
3. Starting with the structure in its geometric outline, remove the member to be cambered.
4. Allow the dead-load force in the member to fall to zero, thereby changing its shape to that corresponding to the no-load condition.
5. Further change the shape of the member to correspond to that at the shop temperature.
6. Accomplish any rounding of member dimensions that may be needed for practical purposes.
7. The total change of shape of the member—from geometric (at normal temperature) to no-load at shop temperature—constitutes the member camber.

It should be noted that the gusset plates for bridge-truss joints are always fabricated with the connecting-member axes coming in at their *geometric* angles. As the members are erected and the joints fitted up, secondary bending moments will be induced at the truss joints under the steel-load-only condition; but these secondary moments will disappear when the bridge reaches its full-dead-load condition.

1.12 Two General Approaches to Fabrication and Erection of Bridge Steelwork

As has been stated previously, the objective in steel bridge construction is to fabricate and erect the structure so that it will have the geometry and stressing designated on the design plans, under full dead load at normal temperature. This geometry is known as the geometric outline. In the case of steel bridges there have been, over the decades, two general procedures for achieving this objective:

1. The "field adjustment" procedure—Carry out a continuing program of steelwork surveys and measurements in the field as erection progresses, in an attempt to discover fabrication and erection deficiencies; and perform continuing steelwork adjustments in an effort to compensate for such deficiencies and for errors in span baselines and pier elevations.
2. The "shop-control" procedure—Place total reliance on first-order surveying of span baselines and pier elevations, and on accurate steelwork fabrication and erection augmented by meticulous construction engineering; and proceed with erection without any field adjustments, on the basis that the resulting bridge dead-load geometry and stressing will be as good as can possibly be achieved.

Bridge designers have a strong tendency to overestimate the capability of field forces to accomplish accurate measurements and effective adjustments of the partially erected structure, and at the same time they tend to underestimate the positive effects of precise steel bridgework fabrication and erection. As a result, we continue to find contract drawings for major steel bridges that call for field evolutions such as the following:

1. Continuous trusses and girders—At the designated stages, measure or "weigh" the reactions on each pier, compare them with calculated theoretical values, and add or remove bearing-shoe shims to bring measured values into agreement with calculated values.
2. Arch bridges—With the arch ribs erected to midspan and only the short, closing "crown sections" not yet in place, measure thrust and moment at the crown, compare them with calculated theoretical values, and then adjust the shape of the closing sections to correct for errors in span-length measurements and in bearing-surface angles at skewback supports, along with accumulated fabrication and erection errors.
3. Suspension bridges—Following erection of the first cable wire or strand across the spans from anchorage to anchorage, survey its sag in each span and adjust these sags to agree with calculated theoretical values.
4. Arch bridges and suspension bridges—Carry out a deck-profile survey along each side of the bridge under the steel-load-only condition, compare survey results with the theoretical profile, and shim the suspender sockets so as to render the bridge floor beams level in the completed structure.
5. Cable-stayed bridges—At each deck-steelwork erection stage, adjust tensions in the newly erected cable stays so as to bring the surveyed deck profile and measured stay tensions into agreement with calculated theoretical data.

There are two prime obstacles to the success of "field adjustment" procedures of whatever type: (1) field determination of the actual geometric and stress conditions of the partially erected structure and its components will not necessarily be definitive, and (2) calculation of the corresponding "proper" or "target" theoretical geometric and stress conditions will most likely prove to be less than authoritative.

1.13 Example of Arch Bridge Construction

In the case of the arch bridge closing sections referred to heretofore, experience on the construction of two major fixed-arch bridges crossing the Niagara River gorge from the United States to Canada—the Rainbow and the Lewiston–Queenston arch bridges (see Figures 1.2 through 1.5)—has demonstrated the difficulty, and indeed the futility, of attempts to make field-measured geometric and stress conditions agree with calculated theoretical values. The broad intent for both structures was to make such adjustments in the shape of the arch-rib closing sections at the crown (which were nominally about 1 ft. [0.3 m] long) as would bring the arch-rib actual crown moments and thrusts into agreement with the calculated theoretical values, thereby correcting for errors in span-length measurements, errors in bearing-surface angles at the skewback supports, and errors in fabrication and erection of the arch-rib sections.

Following extensive theoretical investigations and on-site measurements the steelwork contractor found, in the case of each Niagara arch bridge, that there were large percentage differences between the field-measured and the calculated theoretical values of arch-rib thrust, moment, and line-of-thrust position, and that the measurements could not be interpreted so as to indicate what corrections to the theoretical closing crown sections, if any, should be made. Accordingly, the contractor concluded that the best solution in each case was to abandon any attempts at correction and simply install the theoretical-shape closing crown sections. In each case, the contractor's recommendation was accepted by the design engineer.

Points to be noted in respect to these field-closure evolutions for the two long-span arch bridges are that accurate jack-load closure measurements at the crown are difficult to obtain under field conditions; and calculation of corresponding theoretical crown thrusts and moments are likely to be questionable because of uncertainties in the dead loading, in the weights of erection equipment, and in the steelwork temperature. Therefore, attempts to adjust the shape of the closing crown sections so as to bring the

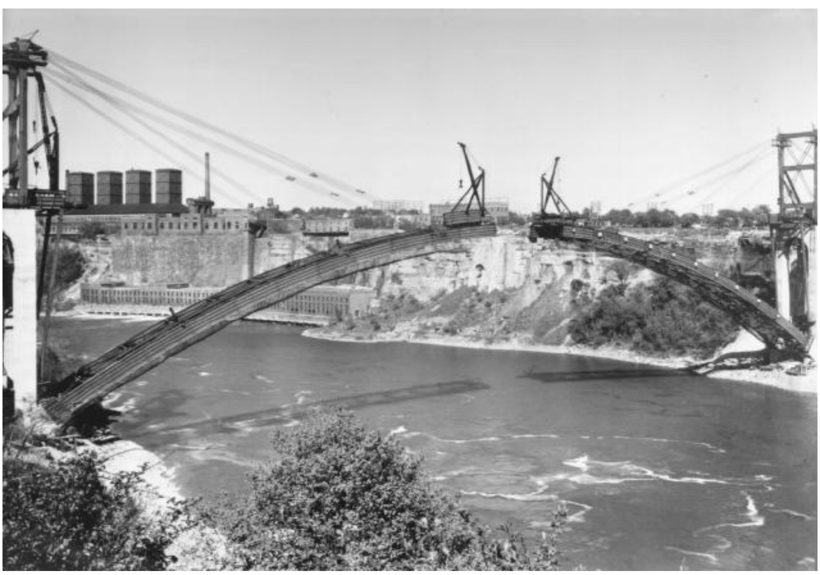


FIGURE 1.2 Erection of arch ribs, Rainbow Bridge, Niagara Falls, New York, 1941. Bridge span is 950 ft. (290 m), with rise of 150 ft. (46 m); box ribs are 3 × 12 ft. (0.91 × 3.66 m). Tiebacks were attached starting at the end of the third tier and jumped forward as erection progressed (see Figure 1.3). Much permanent steelwork was used in tieback bents. Derricks on approaches load steelwork onto material cars that travel up arch ribs. Travelers are shown erecting last full-length arch-rib sections, leaving only the short, closing crown sections to be erected. Canada is at right, the United States at left. (Courtesy of Bethlehem Steel Corporation.)

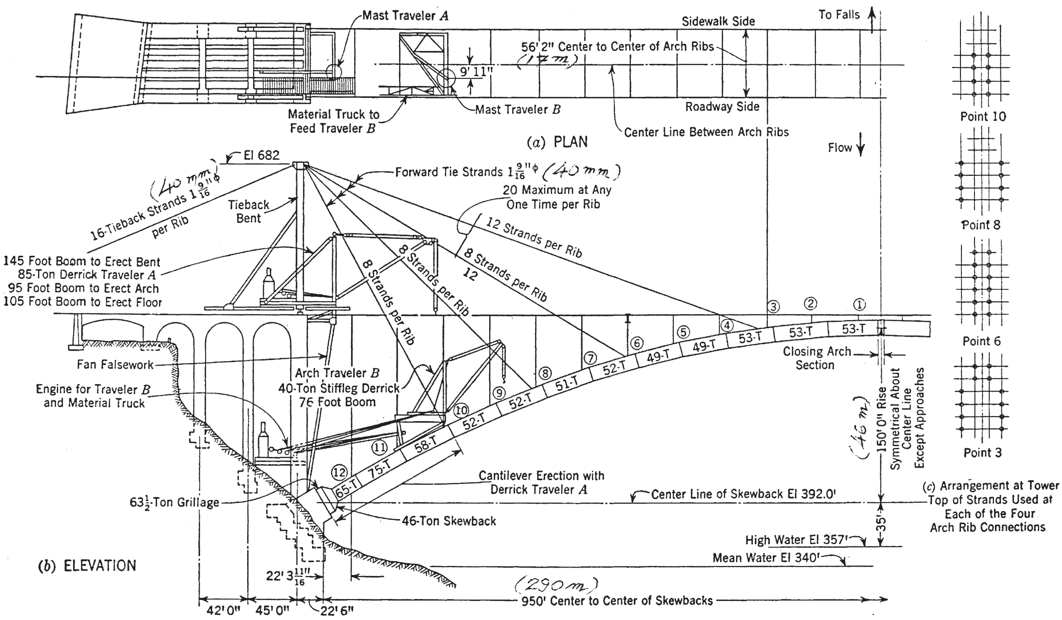


FIGURE 1.3 Rainbow Bridge, Niagara Falls, New York, showing successive arch tieback positions (Durkee, 1945). Arch-rib erection geometry and stressing were controlled by measured tieback tensions in combination with surveyed arch-rib elevations.

FIGURE 1.4 Lewiston–Queenston arch bridge, near Niagara Falls, New York, 1962. The longest fixed-arch span in the United States at 1000 ft. (305 m); rise is 159 ft. (48 m). Box arch-rib sections are typically about 3 × 13 1/2 ft. (0.9 × 4.1 m) in cross-section and about 44 1/2 ft. (13.6 m) long. Job was estimated using erection tiebacks (same as shown in Figure 1.3), but subsequent studies showed the long, sloping falsework bents to be more economical (even if less secure looking). Much permanent steelwork was used in the falsework bents. Derricks on approaches load steelwork onto material cars that travel up arch ribs. The 115-ton-capacity travelers are shown erecting the last full-length arch-rib sections, leaving only the short, closing crown sections to be erected. Canada is at left, the United States at right. (Courtesy of Bethlehem Steel Corporation.)

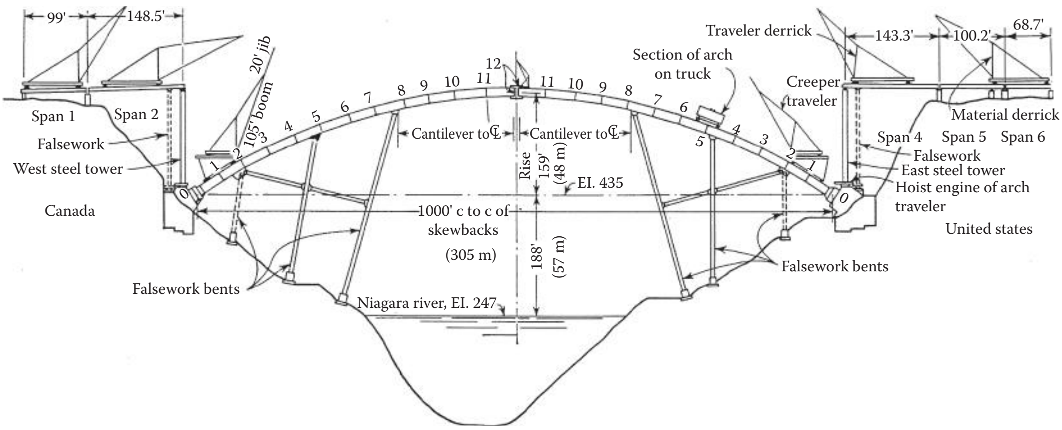


FIGURE 1.5 Lewiston–Queenston arch bridge near Niagara Falls, New York. Crawler cranes erect steelwork for spans 1 and 6 and erect material derricks thereon. These derricks erect traveler derricks, which move forward and erect supporting falsework and spans 2, 5, and 4. Traveler derricks erect arch-rib sections 1 and 2 and supporting falsework at each skewback, then set up creeper derricks, which erect arches to midspan.

actual stress condition of the arch ribs closer to the presumed theoretical condition are not likely to be either practical or successful.

It was concluded that for long, flexible arch ribs, the best construction philosophy and practice is (1) to achieve overall geometric control of the structure by performing all field survey work and steelwork fabrication and erection operations to a meticulous degree of accuracy, and then (2) to rely on that overall geometric control to produce a finished structure having the desired stressing and geometry. For the Rainbow arch bridge, these practical construction considerations were set forth definitively by the contractor (Copp, et al. 1945). The contractor's experience for the Lewiston–Queenston arch bridge was similar to that on Rainbow, and was reported—although in considerably less detail—in Feidler, 1962.

1.14 Which Construction Procedure Is to Be Preferred?

The contractor's experience on the construction of the two long-span fixed-arch bridges is set forth at length since it illustrates a key construction theorem that is broadly applicable to the fabrication and erection of steel bridges of all types. This theorem holds that the contractor's best procedure for achieving, in the completed structure, the dead-load geometry and stressing stipulated on the design plans is generally as follows:

1. Determine dead-load stress data for the structure at its geometric outline (under normal temperature), based on accurately calculated weights for all components.
2. Determine the cambered (i.e., "no-load") dimensions of each component. This involves determining the change of shape of each component from the dead-load geometry, as its dead-load stressing is removed and its temperature is changed from normal to the shop temperature (refer to Section 1.11).
3. Fabricate, with all due precision, each structural component to its proper no-load dimensions—except for certain flexible components such as wire rope and strand members, which may require special treatment.
4. Accomplish shop assembly of members and "reaming assembled" of holes in joints, as needed.
5. Carry out comprehensive engineering studies of the structure under erection at each key erection stage, determining corresponding stress and geometric data, and prepare a step-by-step erection procedure plan, incorporating any check measurements that may be necessary or desirable.

6. During the erection program, bring all members and joints to the designated alignment prior to bolting or welding.
7. Enter and connect the final or closing structural components, following the closing procedure plan, without attempting any field measurements thereof or adjustments thereto.

In summary, the key to construction success is to accomplish the field surveys of critical baselines and support elevations with all due precision, perform construction engineering studies comprehensively and shop fabrication accurately, and then carry the erection evolutions through in the field without any second guessing and ill-advised attempts at measurement and adjustment.

It may be noted that no special treatment is accorded to statically indeterminate members; they are fabricated and erected under the same governing considerations applicable to statically determinate members, as set forth earlier. It may be noted further that this general steel bridge construction philosophy does not rule out check measurements altogether, as erection goes forward; under certain special conditions, measurements of stressing and/or geometry at critical erection stages may be necessary or desirable to confirm structural integrity. However, before the erector calls for any such measurements he should make certain that they will prove to be practical and meaningful.

1.15 Example of Suspension Bridge Cable Construction

To illustrate the "shop-control" construction philosophy further, its application to the main cables of the first Wm. Preston Lane, Jr., Memorial Bridge, crossing the Chesapeake Bay in Maryland, completed in 1952 (Figure 1.6), will be described. Suspension bridge cables constitute one of the most difficult bridge erection challenges (Durkee, 1966). Up until "first Chesapeake" the cables of major suspension bridges had been adjusted to the correct position in each span by a sag survey of the first-erected cable wires or strands, using surveying instruments and target rods. However, on first Chesapeake, with its 1600 ft. (488 m) main span, 661 ft. (201 m) side spans, and 450 ft. (137 m) back spans, the steelwork contractor recommended abandoning the standard cable-sag survey and adopting the "setting-to-mark" procedure for positioning the guide strands—a significant new concept in suspension bridge cable construction.

The steelwork contractor's rationale for "setting to marks" was spelled out in a letter to the design engineer (see Figure 1.7). (The complete letter is reproduced because it spells out significant construction

FIGURE 1.6 Suspension spans of first Chesapeake Bay Bridge, Maryland, 1952. Deck steelwork is under erection and is about 50% complete. A typical four-panel through-truss deck section, weighing about 100 tons, is being picked in west side span, and also in east side span in distance. Main span is 1600 ft. (488 m) and side spans are 661 ft. (201 m); towers are 324 ft. (99 m) high. Cables are 14 in. (356 mm) in diameter and are made up of 61 helical bridge strands each (see Figure 1.8).

July 6th, 1951
JJ:MM
C-1756

[To the design engineer]

Gentlemen: Attention of Mr. ____________________

Re: Chesapeake Bay Bridge — Suspension Span Cables

In our studies of the method of cable erection, we have arrived at the conclusion that setting of the guide strands to measured marks, instead of to surveyed sag, is a more satisfactory and more accurate method. Since such a procedure is not in accordance with the specifications, we wish to present for your consideration the reasoning which has led us to this conclusion, and to describe in outline form our proposed method of setting to marks.

On previous major suspension bridges, most of which have been built with parallel-wire instead of helical-strand cables, the thought has evidently been that setting the guide wire or guide strand to a computed sag, varying with the temperature, would be the most accurate method. This is associated with the fact that guide wires were never measured and marked to length. These established methods were carried over when strand-type cables came into use. An added reason may have been the knowledge that a small error in length results in a relatively large error in sag; and on the present structure the length-error to sag-error ratios are 1:2.4 and 1:1.5 for the main span and side spans, respectively.

However, the reading of the sag in the field is a very difficult operation because of the distances involved, the slopes of the side spans and backstays, the fact that even slight wind causes considerable motion to the guide strand, and for other practical reasons. We also believe that even though readings are made on cloudy days or at night, the actual temperature of all portions of the structure which will affect the sag cannot be accurately known. We are convinced that setting the guide strands according to the length marks thereon, which are place under what amount to laboratory or ideal conditions at the manufacturing plant, will produce more accurate results than would field measurement of the sag.

To be specific, consider the case of field determination of sag in the main span, where it is necessary to establish accessible platforms, and an H.I. and a foresight somewhat below the desired sag elevation; and then to sight on the foresight and bring a target, hung from the guide strand, down to the line-of-sight. In the present case it is 1600 ft (488 m) to the foresight and 800 ft (244 m) to the target. Even if the line-of-sight were established just right, it would be only under perfect conditions of temperature and air — if indeed then — that such a survey would be precise. The difficulties are still greater in the side spans and back spans, where inclined lines-of-sight must be established by a series of offset measurements from distant bench marks. There is always the danger, particularly in the present location and at the time now scheduled, that days may be lost in waiting for the right conditions of weather to make an instrument survey feasible.

There is a second factor of doubt involved. The strand is measured under a known stress and at a known modulus, with "mechanical stretch" taken out. It is then reeled to a relatively small diameter and unreeled at the bridge site. Under its own weight, and until the full dead load has been applied, there is an indeterminable loss in mechanical set, or loss of modulus. A strand set to proper sag for the final modulus will accordingly be set too low, and the final cable will be below plan elevation. This possible error can only be on the side that is less desirable. Evidently, also, it could be on the order of 1-1/2 in (40 mm) of sag increase for 1% of temporary reduction in modulus. If the strand were set to sag based on the assumed smaller modulus than will exist for the fully loaded condition, we doubt whether this smaller modulus could be chosen closely enough to ensure that the final sag would be correct. We are assured, however, by our manufacturing plant, that even though the modulus under bare-cable weight may be subject to unknown variation, the modulus which existed at the pre-stressing bed under the measuring tension will be duplicated when this same tension is

FIGURE 1.7 Setting cable guide strands to marks.

reached under dead load. Therefore, if the guide stand is set to measured marks, the doubt as to modulus is eliminated.

A third source of error is temperature. In past practice the sag has been adjusted, by reference to a chart, in accordance with the existing temperature. Granted that the adjustment is made in the early morning (the fog having risen but the sun not), it is hard to conceive that the actual average temperature in 3955 ft (1205 m) of strand will be that recorded by any thermometer. The mainspan sag error is about 0.7 in. (18 mm) per deg C of temperature.

These conditions are all greatly improved at the strand pre-stressing bed. There seems to be no reason to doubt that the guide strands can be measured and marked to an insignificant degree of error, at a stipulated stress and under a well-soaked and determinable temperature. Any errors in sag level must result from something other than the measured length of the guide strand.

There is one indispensable condition which, however, holds for either method of setting. That is, that the total distance from anchorage to anchorage, and the total calculated length of strand under its own-weight stress, must agree within the limits of shimming provided in the anchorages. Therefore, this distance in the field must be checked to close agreement. While the measured length of strand will be calculated with precision, it is interesting to note that in this calculation, it is not essential that the modulus be known with exactness. The important factor is that the strand length under the final deadload stress will be calculated exactly; and since that length is measured under the corresponding average strand stress, knowledge of the modulus is not a consideration. If the modulus at deadload stress is not as assumed, the only effect will be a change of deflection under live load, and this is minor. We emphasize again that the stand length under dead load, and the length as measured in the prestressing bed, will be identical regardless of the modulus.

The calculations for the bare-cable position result in pulled-back positions for the tops of the towers and cable bents, in order to control the unbalanced forces tending to slip the strands in the saddles. These pullback distances may be slightly in error without the slipping forces overcoming friction and thereby becoming apparent. Such errors would affect the final sags of strands set to sag. However, they would have no effect on the final sags of strands set-to-mark at the saddles; these errors change the temporary strand sags only, and under final stress the sags and the shaft leans will be as called for by the design plans.

It sometimes has happened that a tower which at its base is square to the bridge axis, acquires a slight skew as it rises. The amount of this skew has never, so far as we know, been important. If it is disregarded and the guide strands are attached without any compensating change, then the final loading will, with virtual certainty, pull the tower square. All sources of possible maladjustment have now been discussed except one — the errors in the several span lengths at the base of the towers and bents. The intention is to recognize and accept these, by performing the appropriate check measurements; and to correct for them by slipping the guide strands designated amounts through the saddles such that the center-of-saddle mark on the strand will be offset by that same amount from the centerline of the saddle.

If we have left unexplained herein any factor that seems to you to render our procedure questionable, we are anxious to know of it and discuss it with you in the near future; and we will be glad to come to your offices for this purpose. The detailed preparations for observing strand sags would require considerable time, and we are not now doing any work along those lines.

Yours very truly,
Chief Engineer

FIGURE 1.7 (*Continued*) Setting cable guide strands to marks.

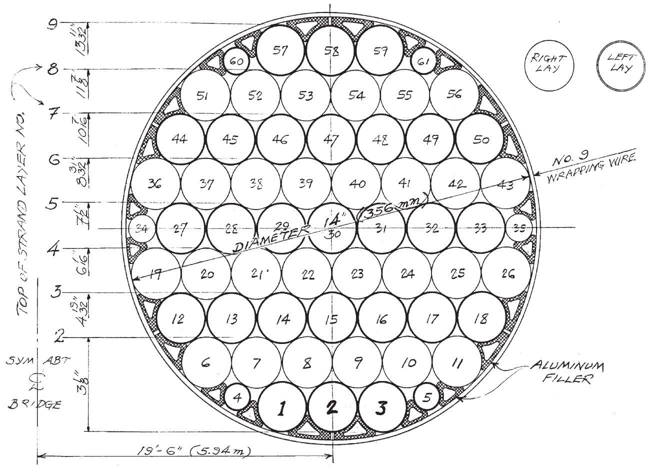


FIGURE 1.8 Main cable of first Chesapeake Bay suspension bridge, Maryland. Each cable consists of 61 helical-type bridge strands, 55 of 1 11/16 in. (43 mm) and 6 of 29/32 in. (23 mm) diameter. Strands 1, 2, and 3 were designated "guide strands" and were set to mark at each saddle and to normal shims at anchorages.

philosophies.) This innovation was accepted by the design engineer. It should be noted that the contractor's major argument was that setting to marks would lead to more accurate cable placement than would a sag survey. The minor arguments, alluded to in the letter, were the resulting savings in preparatory office engineering work and in the field engineering effort, and most likely in construction time as well.

Each cable consisted of 61 standard helical-type bridge strands, as shown in Figure 1.8. To implement the setting-to-mark procedure each of three bottom-layer "guide strands" of each cable (i.e., strands 1, 2, and 3) was accurately measured in the manufacturing shop under the simulated full-dead-load tension, and circumferential marks were placed at the four center-of-saddle positions of each strand. Then, in the field, the guide strands (each about 3955 ft. [1205 m] long) were erected and positioned according to the following procedure:

1. Place the three guide strands for each cable "on the mark" at each of the four saddles and set normal shims at each of the two anchorages.
2. Under conditions of uniform temperature and no wind, measure the sag differences among the three guide strands of each cable, at the center of each of the five spans.
3. Calculate the "center-of-gravity" position for each guide-strand group in each span.
4. Adjust the sag of each strand to bring it to the center-of-gravity position in each span. This position was considered to represent the correct theoretical guide-strand sag in each span.

The maximum "spread" from the highest to the lowest strand at the span center, prior to adjustment, was found to be 1 3/4 in. (44 mm) in the main span, 3 1/2 in. (89 mm) in the side spans, and 3 3/4 in. (95 mm) in the back spans. Further, the maximum change of perpendicular sag needed to bring the guide strands to the center-of-gravity position in each span was found to be 15/16 in. (24 mm) for the main span, 2 1/16 in. (52 mm) for the side spans, and 2 1/16 in. (52 mm) for the back spans. These small adjustments testify to the accuracy of strand fabrication and to the validity of the setting-to-mark strand adjustment procedure, which was declared to be a success by all parties concerned. It seems doubtful that such accuracy in cable positioning could have been achieved using the standard sag-survey procedure.

With the first-layer strands in proper position in each cable, the strands in the second and subsequent layers were positioned to hang correctly in relation to the first layer, as is customary and proper for suspension bridge cable construction.

This example provides good illustration that the construction engineering philosophy referred to as the shop-control procedure can be applied advantageously not only to typical rigid-type steel structures, such as continuous trusses and arches, but also to flexible-type structures, such as suspension bridges.

There is, however, an important caveat: the steelwork contractor must be a firm of suitable caliber and experience.

1.16 Example of Cable-Stayed Bridge Construction

In the case of cable-stayed bridges, the first of which were built in the 1950s, it appears that the governing construction engineering philosophy calls for field measurement and adjustment as the means for control of stay-cable and deck-structure geometry and stressing. For example, we have seen specifications calling for the completed bridge to meet the following geometric and stress requirements:

1. The deck elevation at midspan shall be within 12 in. (305 mm) of theoretical.
2. The deck profile at each cable attachment point shall be within 2 in. (50 mm) of a parabola passing through the actual (i.e., field-measured) midspan point.
3. Cable-stay tensions shall be within 5% of the "corrected theoretical" values.

Such specification requirements introduce a number of problems of interpretation, field measurement, calculation, and field correction procedure, such as the following:

1. Interpretation:
   a. The specifications are silent with respect to transverse elevation differentials. Therefore, two deck-profile control parabolas are presumably needed, one for each side of the bridge.
2. Field measurement of actual deck profile:
   a. The temperature will be neither constant nor uniform throughout the structure during the survey work.
   b. The survey procedure itself will introduce some inherent error.
3. Field measurement of cable-stay tensions:
   a. Hydraulic jacks, if used, are not likely to be accurate within 2%, perhaps even 5%; further, the exact point of "lift off" will be uncertain.
   b. Other procedures for measuring cable tension, such as vibration or strain gaging, do not appear to define tensions within about 5%.
   c. All cable tensions cannot be measured simultaneously; an extended period will be needed, during which conditions will vary and introduce additional errors.
4. Calculation of "actual" bridge profile and cable tensions:
   a. Field-measured data must be transformed by calculation into "corrected actual" bridge profiles and cable tensions, at normal temperature and without erection loads.
   b. Actual dead weights of structural components can differ by perhaps 2% from nominal weights, while temporary erection loads probably cannot be known within about 5%.
   c. The actual temperature of structural components will be uncertain and not uniform.
   d. The mathematical model itself will introduce additional error.
5. "Target condition" of bridge:
   a. The "target condition" to be achieved by field adjustment will differ from the geometric condition, because of the absence of the deck wearing surface and other such components; it must therefore be calculated, introducing additional error.

6. Determining field corrections to be carried out by erector, to transform "corrected actual" bridge into "target condition" bridge:
   a. The bridge structure is highly redundant, and changing any one cable tension will send geometric and cable-tension changes throughout the structure. Thus, an iterative correction procedure will be needed.

It seems likely that the total effect of all these practical factors could easily be sufficient to render ineffective the contractor's attempts to fine-tune the geometry and stressing of the as-erected structure to bring it into agreement with the calculated bridge target condition. Further, there can be no assurance that the specifications requirements for the deck-profile geometry and cable-stay tensions are even compatible; it seems likely that *either* the deck geometry or the cable tensions may be achieved, but not *both*.

Specifications clauses of the type cited seem clearly to constitute unwarranted and unnecessary field- adjustment requirements. Such clauses are typically set forth by bridge designers who have great confidence in computer-generated calculation, but do not have a sufficient background in and understanding of the practical factors associated with steel bridge construction. Experience has shown that field procedures for major bridges developed unilaterally by design engineers should be reviewed carefully to determine whether they are practical and desirable and will in fact achieve the desired objectives.

In view of all these considerations, the question comes forward as to what design and construction principles should be followed to ensure that the dead-load geometry and stressing of steel cable-stayed bridges will fall within acceptable limits. Consistent with the general construction engineering procedures recommended for other types of bridges, we should abandon reliance on field measurements followed by adjustments of geometry and stressing, and instead place prime reliance on proper geometric control of bridge components during fabrication, followed by accurate erection evolutions as the work goes forward in the field.

Accordingly, the proper construction procedure for cable-stayed steel bridges can be summarized as follows:

1. Determine the actual bridge baseline lengths and pier-top elevations to a high degree of accuracy.
2. Fabricate the bridge towers, cables, and girders to a high degree of geometric precision.
3. Determine, in the fabricating shop, the final residual errors in critical fabricated dimensions, including cable-stay lengths after socketing, and positions of socket bearing surfaces or pinholes.
4. Determine "corrected theoretical" positioning for each individual cable stay.
5. During erection, bring all tower and girder structural joints into shop-fabricated alignment, with fair holes, and so on.
6. At the appropriate erection stages, install "corrected theoretical" positional for each cable stay.
7. With the structure in the all-steel-erected condition (or other appropriate designated condition), check it over carefully to determine whether any significant geometric or other discrepancies are in evidence. If there are none, declare conditions acceptable and continue with erection.

This construction engineering philosophy can be summarized by stating that if the steelwork fabrication and erection are properly engineered and carried out, the geometry and stressing of the completed structure will fall within acceptable limits; whereas, if the fabrication and erection are not properly done, corrective measurements and adjustments attempted in the field are not likely to improve the structure, or even to prove satisfactory. Accordingly, in constructing steel cable-stayed bridges we should place full reliance on accurate shop fabrication and on controlled field erection, just as is done on other types of steel bridges, rather than attempting to make measurements and adjustments in the field to compensate for inadequate fabrication and erection Figures 1.9 and 1.10 show girder erections of cable-stayed orthotropic-steel-deck bridge over Mississippi River at Luling.

FIGURE 1.9 Cable-stayed orthotropic-steel-deck bridge over Mississippi River at Luling, Louisiana, 1982; view looking northeast. The main span is 1222 ft. (372 m); the A-frame towers are 350 ft. (107 m) high. A barge-mounted ringer derrick erected the main steelwork, using a 340 ft. (104 m) boom with a 120 ft. (37 m) jib to erect tower components weighing up to 183 tons, and using a shorter boom for deck components. Cable stays at the ends of projecting cross girders are permanent; others are temporary erection stays. Girder section 16-west of north portion of bridge, erected a few days previously, is projecting at left; companion girder section 16-east is on barge ready for erection (see Figure 1.10).

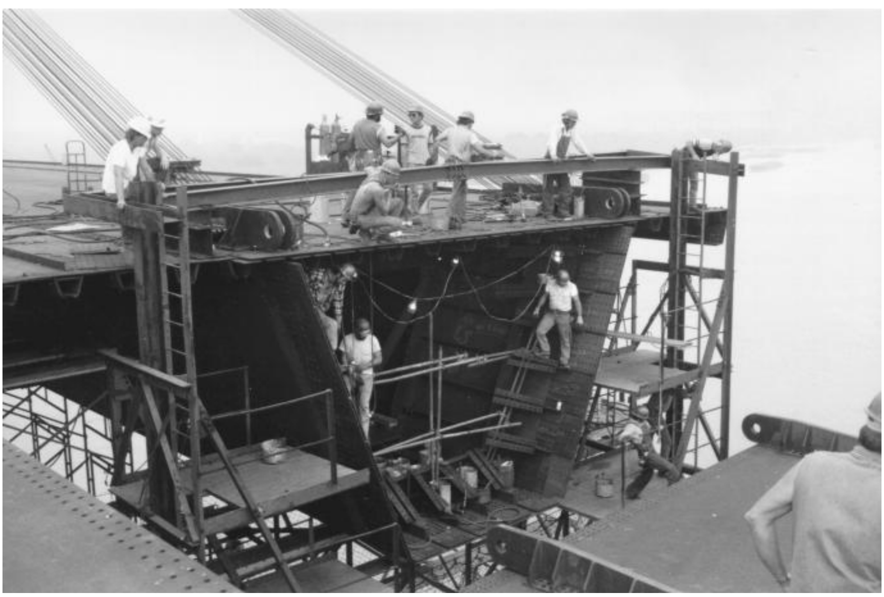


FIGURE 1.10 Luling Bridge deck steelwork erection, 1982; view looking northeast (refer to Figure 1.9). The twin box girders are 14 ft. (4.3 m) deep; the deck plate is 7/16 in. (11 mm) thick. Girder section 16-east is being raised into position (lower right) and will be secured by large-pin hinge bars prior to fairing-up of joint holes and permanent bolting. Temporary erection stays are jumped forward as girder erection progresses.

1.17 Field Checking at Critical Erection Stages

As has been stated previously, the best governing procedure for steel bridge construction is generally the shop-control procedure, wherein full reliance is placed on accurate fabrication of the bridge components as the basis for the integrity of the completed structure. However, this philosophy does not rule out the desirability of certain checks in the field as erection goes forward, with the objective of providing assurance that the work is on target and no significant errors have been introduced.

It would be impossible to catalog those cases during steel bridge construction where a field check might be desirable; such cases will generally suggest themselves as the construction engineering studies progress. We will only comment that these field-check cases, and the procedures to be used, should be looked at carefully, and even skeptically, to make certain that the measurements will be both desirable and practical, producing meaningful information that can be used to augment job integrity.

1.18 Determination of Erection-Strength Adequacy

Quite commonly, bridge member forces during the erection stages will be altogether different from those that will prevail in the completed structure. At each critical erection stage, the bridge members must be reviewed for strength and stability, to ensure structural integrity as the work goes forward. Such a construction engineering review is typically the responsibility of the steelwork erector, who carries out thorough erection studies of the structure and calls for strengthening or stabilizing of members as needed. The erector submits the studies and recommendations to the design engineer for review and comment, but normally the full responsibility for steelwork structural integrity during erection rests with the erector.

In the United States, bridgework design specifications commonly require that stresses in steel structures under erection shall not exceed certain multiples of design allowable stresses. Although this type of erection stress limitation is probably safe for most steel structures under ordinary conditions, it is not necessarily adequate for the control of the erection stressing of large monumental-type bridges. The key point to be understood here is that fundamentally, there is no logical fixed relationship between design allowable stresses, which are based on somewhat uncertain long-term service loading requirements along with some degree of assumed structural deterioration, and stresses that are safe and economical during the bridge erection stages, where loads and their locations are normally well defined and the structural material is in new condition. Clearly, the basic premises of the two situations are significantly different, and “factored design stresses” must therefore be considered unreliable as a basis for evaluating erection safety.

There is yet a further problem with factored design stresses. Large truss-type bridges in various erection stages may undergo deflections and distortions that are substantial compared with those occurring under service conditions, thereby introducing apprehension regarding the effect of the secondary bending stresses that result from joint rigidity.

Recognizing these basic considerations, the engineering department of a major U.S. steelwork contractor went forward in the early 1970s to develop a logical philosophy for erection-strength appraisal of large structural steel frameworks, with particular reference to long-span bridges, and implemented this philosophy with a stress analysis procedure. The effort was successful and the results were reported in a paper published by the American Society of Civil Engineers in 1977 (Durkee Thomaides, 1977). This stress analysis procedure, designated the erection rating factor (ERF) procedure, is founded directly on basic structural principles, rather than on bridge-member design specifications, which are essentially irrelevant to the problem of erection stressing.

It may be noted that a significant inducement toward development of the ERF procedure was the failure of the first Quebec cantilever bridge in 1907 (see Figures 1.11 and 1.12). It was quite obvious that evaluation of the structural safety of the Quebec bridge at advanced cantilever erection stages such as that portrayed in Figure 1.11, by the factored-design-stress procedure, would inspire no confidence and would not be justifiable.

FIGURE 1.11 First Quebec railway cantilever bridge, August 23, 1907. Cantilever erection of south main span, 6 days before collapse. The tower traveler erected the anchor span (on falsework) and then the cantilever arm; then erected the top-chord traveler, which is shown erecting suspended span at end of cantilever arm. The main span of 1800 ft. (549 m) was the world's longest of any type. The sidespan bottom chords second from pier failed in compression because latticing connecting chord corner angles was deficient under secondary bending conditions.

FIGURE 1.12 Wreckage of south anchor span of first Quebec railway cantilever bridge, 1907. View looking north from south shore a few days after collapse of August 29, 1907, the worst disaster in the history of bridge construction. About 20,000 tons of steelwork fell into the St. Lawrence River, and 75 workmen lost their lives.

The ERF procedure for a truss bridge can be summarized as follows:

1. Assume either (1) pin-ended members (no secondary bending), (2) plane-frame action (rigid truss joints, secondary bending in one plane), or (3) space-frame action (bracing-member joints also rigid, secondary bending in two planes), as engineering judgment dictates.
2. Determine, for each designated erection stage, the member primary forces (axial) and secondary forces (bending) attributable to gravity loads and wind loads.
3. Compute the member stresses induced by the combined erection axial forces and bending moments.
4. Compute the ERF for each member at three or five locations: at the middle of the member; at each joint, inside the gusset plates (usually at the first row of bolts); and, where upset member plates or gusset plates are used, at the stepped-down cross section outside each joint.
5. Determine the minimum computed ERF for each member and compare it with the stipulated minimum value.
6. Where the computed minimum ERF equals or exceeds the stipulated minimum value, the member is considered satisfactory. Where it is less, the member may be inadequate; reevaluate the critical part of it in greater detail and recalculate the ERF for further comparison with the stipulated minimum. (Initially calculated values can often be increased significantly.)
7. Where the computed minimum ERF remains less than the stipulated minimum value, strengthen the member as required.

Note that member forces attributable to wind are treated the same as those attributable to gravity loads. The old concept of "increased allowable stresses" for wind is not considered to be valid for erection conditions and is not used in the ERF procedure. Maximum acceptable *l/r* and *b/t* values are included in the criteria. ERFs for members subjected to secondary bending moments are calculated using interaction equations.

1.19 Philosophy of the Erection Rating Factor

In order that the structural integrity and reliability of a steel framework can be maintained throughout the erection program, the minimum probable (or "minimum characteristic") strength value of each member must necessarily be no less than the maximum probable (or "maximum characteristic") force value, under the most adverse erection condition. In other words, the following relationship is required:

$$S - \Delta S \geq F + \Delta F \tag{1.1}$$

where S is the computed or nominal strength value for the member; ΔS the maximum probable member strength underrun from the computed or nominal value; F the computed or nominal force value for the member; and ΔF the maximum probable member force overrun from the computed or nominal value.

Equation 1.1 states that in the event the actual strength of the structural member is less than the nominal strength, S, by an amount ΔS, while at same time the actual force in the member is greater than the nominal force, F, by an amount ΔF, the member strength will still be no less than the member force, and so the member will not fail during erection. This equation provides a direct appraisal of erection realities, in contrast to the allowable-stress approach based on factored design stresses.

Proceeding now to rearrange the terms in Equation 1.1, we find that

$$S\left(1 - \frac{\Delta S}{S}\right) \geq F\left(1 + \frac{\Delta F}{F}\right); \quad \frac{S}{F} \geq \frac{1 + (\Delta F/F)}{1 - (\Delta S/S)} \tag{1.2}$$

The ERF is now defined as

$$\text{ERF} \equiv \frac{S}{F} \tag{1.3}$$

that is, the nominal strength value, S, of the member divided by its nominal force value, F. Thus, for erection structural integrity and reliability to be maintained, it is necessary that

$$\mathrm{ERF} \geq \frac{1+(\Delta F/F)}{1-(\Delta S/S)} \tag{1.4}$$

1.20 Minimum Erection Rating Factors

In view of possible errors in (1) the assumed weight of permanent structural components, (2) the assumed weight and positioning of erection equipment, and (3) the mathematical models assumed for purposes of erection structural analysis, it is reasonable to assume that the actual member force for a given erection condition may exceed the computed force value by as much as 10%; that is, it is reasonable to take $\Delta F/F$ as equal to 0.10.

For tension members, uncertainties in (1) the area of the cross section, (2) the strength of the material, and (3) the member workmanship, indicate that the actual member strength may be up to 15% less than the computed value; that is, $\Delta S/S$ can reasonably be taken as equal to 0.15. The additional uncertainties associated with compression member strength suggest that $\Delta S/S$ be taken as 0.25 for those members. Placing these values into Equation 1.4, we obtain the following minimum ERFs:

Tension members:

$$\mathrm{ERF}_{\mathrm{tmin}} = \frac{(1+0.10)}{(1-0.15)} = 1.294, \quad \text{say } 1.30$$

Compression member:

$$\mathrm{ERF}_{\mathrm{cmin}} = \frac{(1+0.10)}{(1-0.25)} = 1.467, \quad \text{say } 1.45$$

The proper interpretation of these expressions is that if, for a given tension (compression) member, the ERF is calculated as 1.30 (1.45) or more, the member can be declared safe for the particular erection condition. Note that higher, or lower, values of ERFs may be selected if conditions warrant.

The minimum ERFs determined as indicated are based on experience and judgment, guided by analysis and test results. They do not reflect any specific probabilities of failure and thus are not based on the concept of an acceptable risk of failure, which might be considered the key to a totally rational approach to structural safety. This possible shortcoming in the ERF procedure might be at least partially overcome by evaluating the parameters $\Delta F/F$ and $\Delta S/S$ on a statistical basis; however, this would involve a considerable effort, and it might not even produce significant results.

It is important to recognize that the ERF procedure for determining erection-strength adequacy is based directly on fundamental strength and stability criteria, rather than being only indirectly related to such criteria through the medium of a design specification. Thus, the procedure gives uniform results for the erection rating of framed structural members irrespective of the specification that was used to design the members. Obviously, the end use of the completed structure is irrelevant to its strength adequacy during the erection configurations, and therefore the design specification should not be brought into the picture as the basis for erection appraisal.

Experience with application of the ERF procedure to long-span truss bridges has shown that it places the erection engineer in much better contact with the physical significance of the analysis than can be obtained by using the factored-design-stress procedure. Further, the ERF procedure takes account of secondary stresses, which have generally been neglected in erection stress analysis.

Although the ERF procedure was prepared for application to truss bridge members, the simple governing structural principle set forth by Equation 1.1 could readily be applied to bridge members and components of any type.

1.21 Deficiencies of Typical Construction Procedure Drawings and Instructions

At this stage of the review it is appropriate to bring forward a key problem in the realm of bridge construction engineering: the strong tendency for construction procedure drawings to be insufficiently clear, and for step-by-step instructions to be either lacking or less than definitive. As a result of these deficiencies it is not uncommon to find the contractor's shop and field evolutions to be going along under something less than suitable control.

Shop and field operations personnel who are in a position to speak frankly to construction engineers will sometimes let them know that procedure drawings and instructions often need to be clarified and upgraded (Figure 1.13). This is a pervasive problem, and it results from two prime causes: (1) the fabrication and erection engineers responsible for drawings and instructions do not have adequate on-the-job experience, and (2) they are not sufficiently skilled in the art of setting forth on the documents, clearly and concisely, exactly what is to be done by the operations forces—and, sometimes of equal importance, what *is not* to be done.

This matter of clear and concise construction procedure drawings and instructions may appear to be a pedestrian matter, but it is decidedly not. *It is a key issue of utmost importance to the success of steel bridge construction.*

1.22 Shop and Field Liaison by Construction Engineers

In addition to the need for well-prepared construction procedure drawings and instructions, it is essential for the staff engineers carrying out construction engineering to set up good working relations with the shop and field production forces, and to visit the work sites and establish effective communication with the personnel responsible for accomplishing what is shown on the documents.

Construction engineers should review each projected operation in detail with the work forces, and upgrade the procedure drawings and instructions as necessary, as the work goes forward. Further,

FIGURE 1.13 Visiting the work site. It is of first-order importance for bridge construction engineers to visit the site regularly and confer with the job superintendent and his foremen regarding practical considerations. Construction engineers have much to learn from the work forces in shop and field, and vice versa. (Courtesy of Bethlehem Steel Corporation.)

engineers should be present at the work sites during critical stages of fabrication and erection. As a component of these site visits, the engineers should organize special meetings of key production personnel to go over critical operations in detail—complete with slides and blackboard as needed— thereby providing the work forces with opportunities to ask questions and discuss procedures and potential problems, and providing engineers the opportunity to determine how well the work forces understand the operations to be carried out.

This matter of liaison between the office and the work sites—like the preceding issue of clear construction procedure documents—may appear to be somewhat prosaic; again, however, it *is a matter of paramount importance*. Failure to attend to these two key issues constitutes a serious problem in steel bridge construction, and opens the door to high costs and delays, and even to erection accidents.

1.23 Comprehensive Bridge Erection-Engineering Specifications

The ERF procedure for determination of erection-strength adequacy, as set forth heretofore for bridge trusswork, could readily be extended to cover bridge members and components of any type under erection loading conditions. Bridge construction engineers should work toward this objective, to release erection-strength appraisal from the limitations of the commonly used factored-design-stress procedure.

Looking still further ahead, it is apparent that there is need in the bridge engineering profession for comprehensive erection-engineering specifications for steel bridge construction. Such specifications should include guidelines for such matters as devising and evaluating erection schemes, determining erection loads, evaluating erection-strength adequacy of all types of bridge members and components, designing erection equipment, and designing temporary erection members such as falsework, tie-downs, tiebacks, and jacking struts. The specifications might also cover contractual considerations associated with construction engineering.

The key point to be recognized here is that the use of bridge *design* specifications as the basis for erection-engineering studies, as is currently the custom, is not appropriate. Erection engineering is a related but different discipline, and should have its own specifications. However, given the current fragmented state of construction engineering in the United States (refer to Section 1.26), it is difficult to envision how such erection-engineering specifications could be prepared. Proprietary considerations associated with each erection firm's experience and procedures could constitute an additional obstacle.

1.24 Standard Conditions for Contracting

A further basic problem in respect to the future of steel bridge construction in the United States lies in the absence of standard conditions for contracting.

On through the nineteenth century both the design and the construction of a major bridge in the United States were frequently the responsibility of a single prominent engineer, who could readily direct and coordinate the work and resolve problems equitably. Then, over, the first 30 years or so of the twentieth century this system was progressively displaced by the practice of competitive bidding on plans and specifications prepared by a design engineer retained by the owner. As a result the responsibility for the structure previously carried by the designer–builder became divided, with the designer taking responsibility for service integrity of the completed structure while prime responsibility for structural adequacy and safety during construction was assumed by the contractor. Full control over the preparation of the plans and specifications—the contract documents—was retained by the design engineer.

This divided responsibility has resulted in contract documents that may not be altogether equitable, since the designer is inevitably under pressure to look after the immediate financial interests of his client, the owner. Documents prepared by only one party to a contract can hardly be expected to reflect the appropriate interest of the other party. However, until about mid-twentieth century, design and construction responsibilities for major bridgework, although divided between the design engineer and the construction engineer, were nonetheless usually under the control of leading members of the bridge engineering profession who were able to command the level of communication and cooperation needed for resolution of inevitable differences of opinion within a framework of equity and good will.

Since the 1970s there has been a trend away from this traditional system of control. The business and management aspects of design firms have become increasingly important, while at the same time steelwork construction firms have become more oriented toward commercial and legal considerations. Professional design and construction engineers have lost stature correspondingly. As a result of these adverse trends, bridgework specifications are being ever more stringently drawn, bidding practices are becoming increasingly aggressive, claims for extra reimbursement are proliferating, insurance costs for all concerned are rising, and control of bridge engineering and construction is being influenced to an increasing extent by administrators and attorneys. These developments have not benefited the bridge owners, the design engineering profession, the steelwork construction industry, or the public—which must ultimately pay all of the costs of bridge construction.

It seems clear that to move forward out of this unsatisfactory state of affairs, a comprehensive set of standard conditions for contracting should be developed to serve as a core document for civil engineering construction—a document that would require only the addition of special provisions to constitute the basic specifications for any major bridge construction project. Such standard conditions would have to be prepared "off line" by a group of high-level engineering delegates having well-established engineering credentials.

A core contract document such as the one proposed has been in general use in Great Britain since 1945, when the first edition of the *Conditions of Contract and Forms of Tender, Agreement and Bond for Use in Connection with Works of Civil Engineering Construction* was published by The Institution of Civil Engineers (ICE). This document, known informally as *The ICE Conditions of Contract* is now in its sixth edition (ICE 2003). It is kept under review and revised as necessary by a permanent Joint Contracts Committee consisting of delegates from ICE, The Association of Consulting Engineers, and The Federation of Civil Engineering Contractors. This document is used as the basis for the majority of works of civil engineering construction that are contracted in Great Britain, including steel bridges.

Further comments on the perceived need for U.S. standard conditions for contracting can be found in Durkee (1977).

1.25 Design and Construct

As has been mentioned, design and construct was common practice in the United States during the nineteenth century. Probably the most notable example was the Brooklyn Bridge, where the designer–builders were John A. Roebling and his son Washington A. Roebling. Construction of the Brooklyn Bridge was begun in 1869 and completed in 1883. Design and construct continued in use through the early years of the twentieth century; the most prominent example from that era may be the Ambassador suspension bridge between Detroit, Michigan and Windsor, Ontario, Canada, completed in 1929. The Ambassador Bridge was designed and built by the McClintic-Marshall Construction Co., Jonathan Jones, chief engineer; it has an 1850 ft. main span, at that time the world's record single span.

Design and construct has not been used for a major steel bridge in the United States since the Ambassador Bridge. However, the procedure has seen significant use throughout the twentieth century for bridges in other countries and particularly in Europe; and most recently design–construct–operate–maintain has come into the picture. Whether these procedures will find significant application in the United States remains to be seen.

The advantages of design and construct are readily apparent:

1. More prospective designs are likely to come forward, than when designs are obtained from only a single organization.
2. Competitive designs are submitted at a preliminary level, making it possible for the owner to provide some input to the selected design between the preliminary stage and design completion.
3. The owner knows the price of the project at the time the preliminary design is selected, as compared with design-bid where the price is not known until the design is completed and bids are received.

4. As the project goes forward, the owner deals with only a single entity, thereby reducing and simplifying his administrative effort.
5. The design-and-construct team members must work effectively together, eliminating the antagonisms and confrontations that can occur on a design-bid project.

A key requirement in the design-and-construct system for a project is the meticulous preparation of the request for proposals (RFP). The following essentials should be covered in suitable detail and clarity:

1. Description of project to be constructed
2. Scope of work
3. Structural component types and characteristics: which are required, which are acceptable, and which are not acceptable
4. Minimum percentages of design and construction work that must be performed by the team's own forces
5. Work schedules; time incentives and disincentives
6. Procedure to be followed when actual conditions are found to differ from those assumed
7. Quality control and quality assurance factors
8. Owner's approval prerogatives during final-design stage and construction stage
9. Applicable local, state, and federal regulations
10. Performance and payment bonding requirements
11. Warranty requirements
12. Owner's procedure for final approval of completed project

In preparing the RFP, the owner should muster all necessary resources from both inside and outside his organization. Political considerations should be given due attention. Document drafts should receive the appropriate reviews, and an RFP brought forward that is in near-final condition. Then, at the start of the contracting process, the owner will typically proceed as follows:

1. Announce the project and invite prospective teams to submit qualifications.
2. Prequalify a small number of teams, perhaps three to five, and send the draft RFP to each.
3. Hold a meeting with the prequalified teams for informal exchange of information and to discuss questions.
4. Prepare the final RFP and issue it to each prequalified team, and announce the date on which proposals will be due.

The owner will customarily call for the proposals to be submitted in two separate components: the design component, showing the preliminary design carried to about the 25% level; and the monetary component, stating the lump-sum bid. Before the bids are opened the owner will typically carry out a scoring process for the preliminary designs, not identifying the teams with their designs, using a 10-point or 100-point grading scale and giving consideration to the following factors:

1. Quality of the design
2. Bridge aesthetics
3. Fabrication and erection feasibility and reliability
4. Construction safety aspects
5. Warranty and long-term maintenance considerations
6. User costs

Using these and other such scoring factors (which can be assigned weights if desired), a final overall design score is assigned to each preliminary design. Then the lump-sum bids are opened. A typical procedure is to divide each team's bid price by its design score, yielding and overall price rating, and to award the contract to the design-and-construct team having the lowest price rating.

Following the contract award the successful team will proceed to bring its preliminary design up to the final-design level, with no site work permitted during this interval. It is customary for the owner to award each unsuccessful submitting team a stipend to partially offset the costs of proposal preparation.

1.26 Construction Engineering Procedures and Practices: The Future

The many existing differences of opinion and procedures in respect to proper governance of steelwork fabrication and erection for major steel bridges raise the question: How do proper bridge construction guidelines come into existence and find their way into practice and into bridge specifications? Looking back over the period roughly from 1900 to 1975, we find that the major steelwork construction companies in the United States developed and maintained competent engineering departments that planned and engineered large bridges (and smaller ones as well) through the fabrication and erection processes with a high degree of proficiency. Traditionally, the steelwork contractor's engineers worked in cooperation with design-office engineers to develop the full range of bridgework technical factors, including construction procedure and practices.

However, times have changed; since the 1970s major steel bridge contractors have all but disappeared in the United States, and further, very few bridge design offices have on their staffs engineers experienced in fabrication and erection engineering. As a result, construction engineering often receives less attention and effort than it needs and deserves, and this is not a good omen for the future of the design and construction of large bridges in the United States.

Bridge construction engineering is not a subject that is or can be taught in the classroom; it must be learned on the job with major steelwork contractors. The best route for an aspiring young construction engineer is to spend significant amounts of time in the fabricating shop and at the bridge site, interspersed with time doing construction engineering technical work in the office. It has been pointed out previously that although construction engineering and design engineering are related, they constitute different practices and require diverse backgrounds and experience. Design engineering can essentially be learned in the design office; construction engineering, however, cannot—it requires a background of experience at work sites. Such experience, it may be noted, is valuable also for design engineers; however, it is not as necessary for them as it is for construction engineers.

The training of future steelwork construction engineers in the United States will be handicapped by the demise of the "Big Two" steelwork contractors in the 1970s. Regrettably, it appears that surviving steelwork contractors in the United States generally do not have the resources for supporting strong engineering departments, and so there is some question as to where the next generation of steel bridge construction engineers in the United States will be coming from.

1.27 Concluding Comments

In closing this review of steel bridge construction, it is appropriate to quote from the work of an illustrious British engineer, teacher, and author, the late Sir Alfred Pugsley (1968):

> A further crop of [bridge] accidents arose last century from overloading by traffic of various kinds, but as we have seen, engineers today concentrate much of their effort to ensure that a margin of strength is provided against this eventuality. But there is one type of collapse that occurs almost as frequently today as it has over the centuries: collapse at a late stage of erection.
>
> The erection of a bridge has always presented its special perils and, in spite of ever-increasing care over the centuries, few great bridges have been built without loss of life. Quite apart from the vagaries of human error, with nearly all bridges there comes a critical time near completion when the success of the bridge hinges on some special operation. Among such are the fitting of a last section in a steel arch, the insertion of the closing central [members] in a cantilever bridge, and the lifting of the roadway deck [structure] into position on a suspension bridge. And there have been major accidents in many such cases. It may be wondered why, if such critical circumstances are well known to arise, adequate care is not taken to prevent an accident. Special care is of course taken, but there are often reasons why there may still be "a slip bewixt cup and lip." Such operations commonly involve

unusually close cooperation between constructors and designers, and between every grade of staff, from the laborers to the designers and directors concerned; and this may put a strain on the design skill, on detailed inspection, and on practical leadership that is enough to exhaust even a Brunel.

In such circumstances it does well to recall the dictum that "it is essential not to have faith in human nature. Such faith is a recent heresy and a very disastrous one." One must rely heavily on the lessons of past experience in the profession. Some of this experience is embodied in professional papers describing erection processes, often (and particularly to young engineers) superficially uninteresting. Some is crystallized in organizational habits, such as the appointment of resident engineers from both the contracting and design sides. And some in precautions I have myself endeavored to list.

It is an easy matter to list such precautions and warnings, but quite another for the senior engineers responsible for the completion of a bridge to stand their ground in real life. This is an area of our subject that depends in a very real sense on the personal qualities of bridge engineers. At bottom, the safety of our bridges depends heavily upon the integrity of our engineers, particularly the leading ones.

1.28 Further Illustrations of Bridges under Construction, Showing Erection Methods (Figures 1.14 through 1.46)

FIGURE 1.14 Royal Albert Bridge across River Tamar, Saltash, England, 1857. The two 455 ft. (139 m) main spans, each weighing 1060 tons, were constructed on shore, floated out on pairs of barges, and hoisted about 100 ft. (30 m) to their final position using hydraulic jacks. Pier masonry was built up after each 3 ft. (1 m) lift.

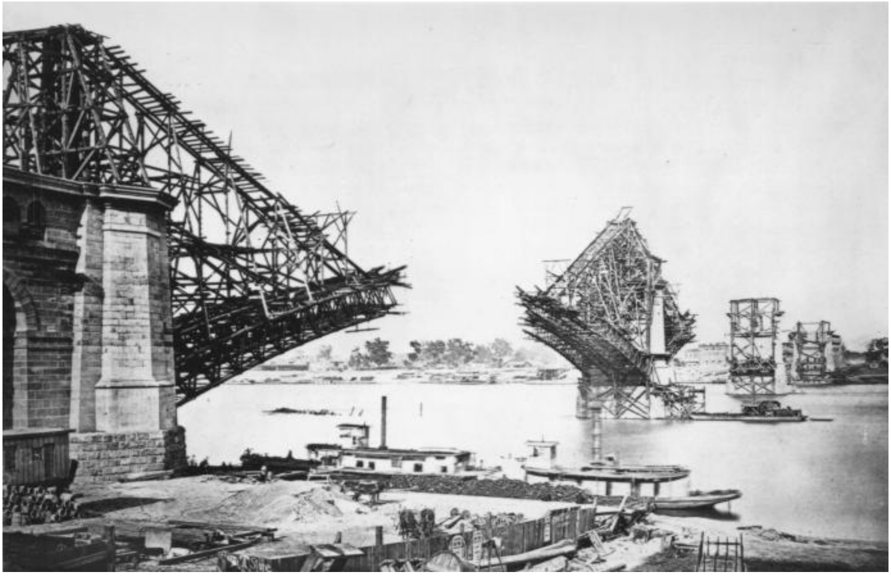


FIGURE 1.15 Eads Bridge across the Mississippi River, St. Louis, Mo., 1873. The first important metal arch bridge in the U.S., it is supported by four planes of hingeless trussed arches having chrome-steel tubular chords. Spans are 502-520-502 ft. (153-158-153 m). During erection, arch trusses were tied back by cables passing over temporary towers built on the piers. Arch ribs were packed in ice to effect closure.

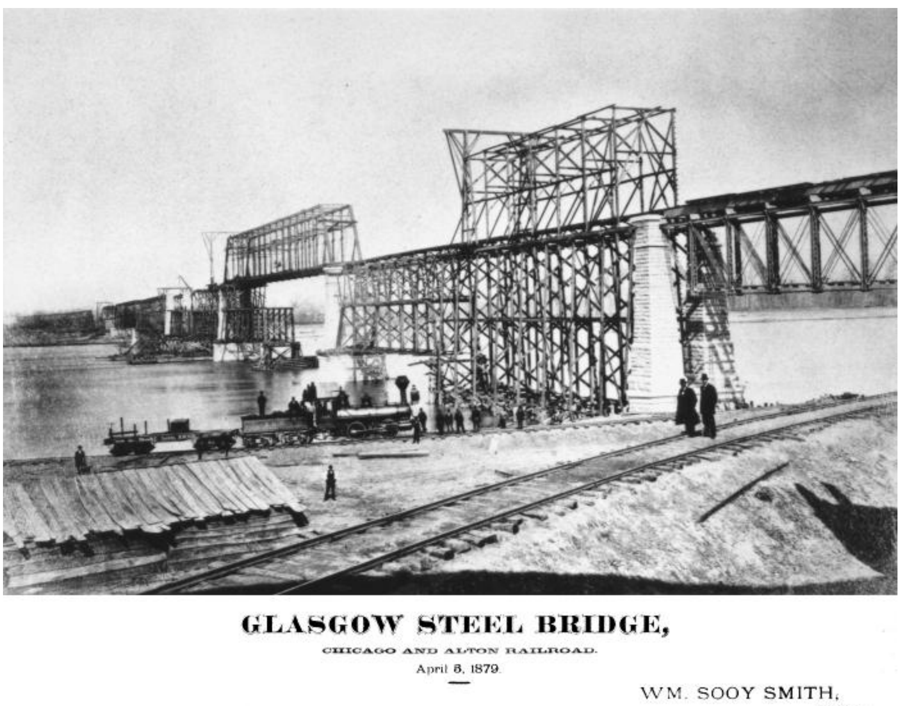


FIGURE 1.16 Glasgow (Missouri) railway truss bridge, 1879. Erection on full supporting falsework was common-place in the 19th century. The world's first all-steel bridge, with five 315 ft. (96 m) through-truss simple spans, crossed the Missouri River.

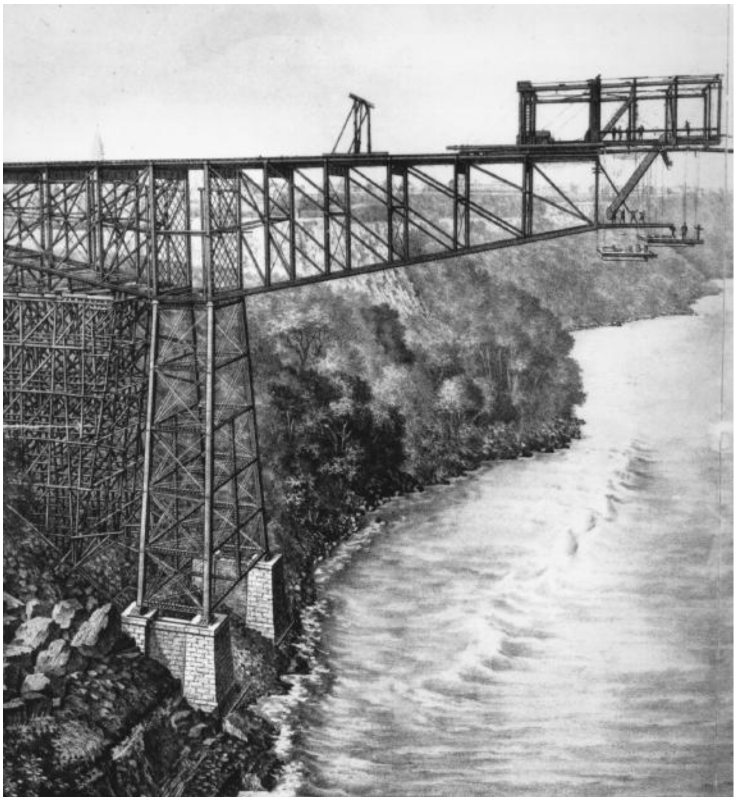
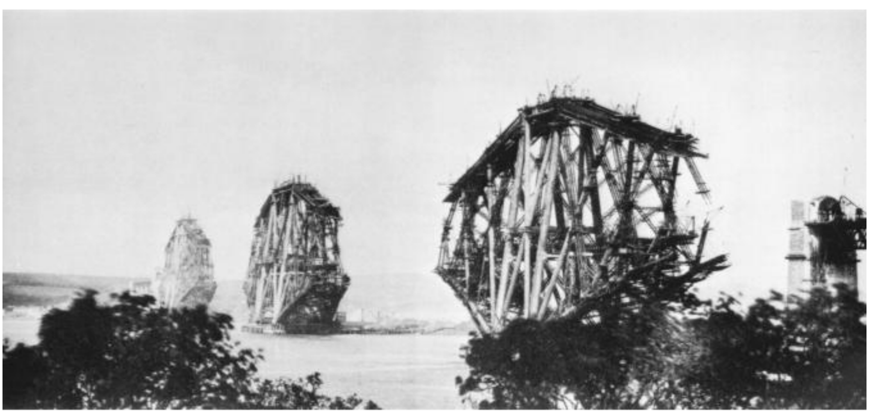


FIGURE 1.17 Niagara River railway cantilever truss bridge, near Niagara Falls, New York, 1883. Massive wood erection traveler constructed side span on falsework, then cantilevered half of main span to midspan. Erection of other half of bridge was similar. First modern-type cantilever bridge, with 470 ft. (143 m) clear main span having a 120 ft. (37 m) center suspended span.



a. The massive cantilevers of the Forth bridge, shown under erection, were conceived in the shadow of the Tay bridge disaster.

FIGURE 1.18 Construction of monumental Forth Bridge, Scotland, 1888. Numerous small movable booms were used, along with erection travelers for cantilevering the two 1710 ft. (521 m) main spans. The main compression members are tubes 12 ft. (3.65 m) in diameter; many other members are also tubular. Total steelwork weight is 51,000 tons. Records are not clear regarding such essentials as cambering and field fitting of individual members in this heavily redundant railway bridge. The Forth is arguably the world's greatest steel structure.

FIGURE 1.19 Pecos River railway viaduct, Texas, 1892. Erection by massive steam-powered wood traveler having many sets of falls and very long reach. Cantilever-truss main span has 185 ft. (56 m) clear opening.

FIGURE 1.20 Raising of suspended span, Carquinez Strait Bridge, California, 1927. The 433 ft. (132 m) suspended span, weighting 650 tons, was raised into position in 35 min., driven by four counterweight boxes having a total weight of 740 tons.

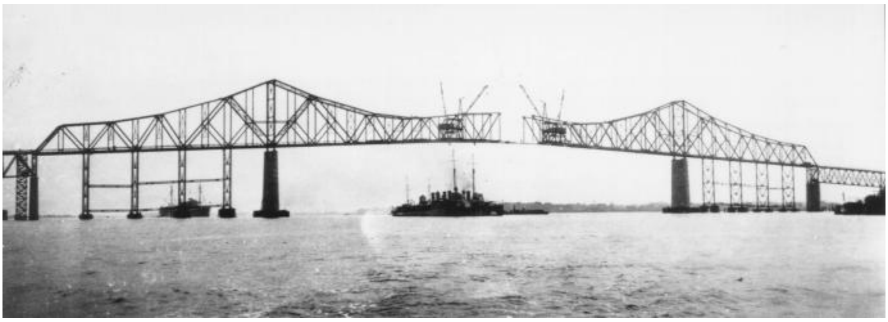


FIGURE 1.21 First Cooper River cantilever bridge, Charleston, S.C., 1929. Erection travelers constructed 450 ft. (137 m) side spans on falsework, then went on to erect 1050 ft. (320 m) main span (including 437.5 ft. [133 m] suspended span) by cantilevering to midspan.

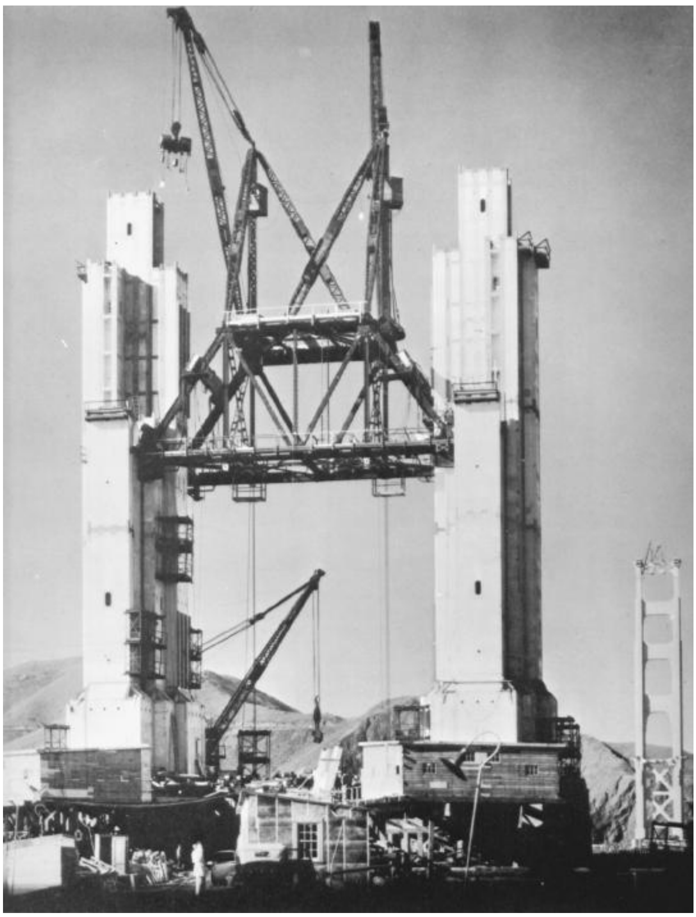
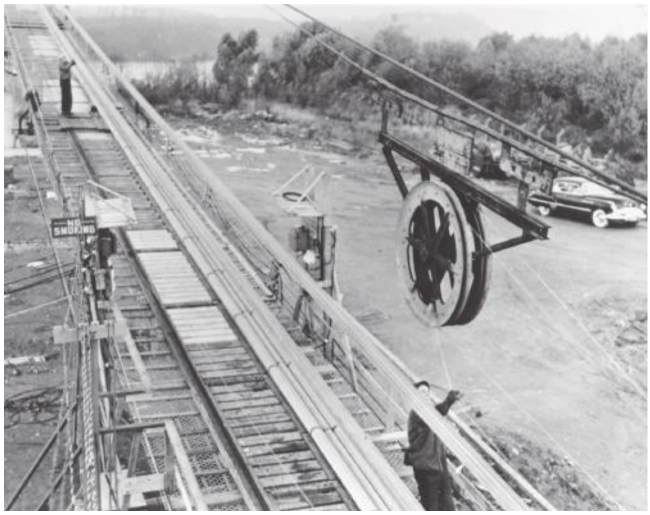


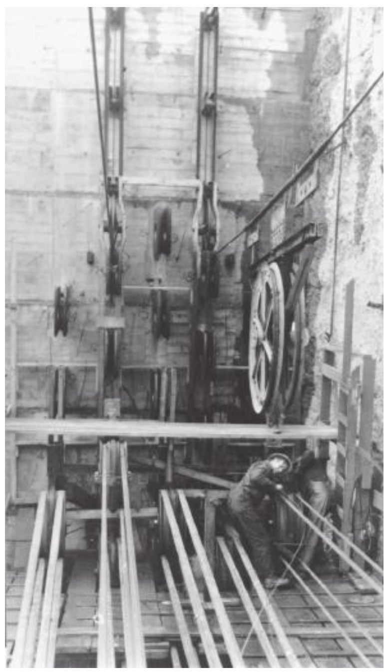
FIGURE 1.22 Erecting south tower of Golden Gate Bridge, San Francisco, 1935. A creeper traveler with two 90 ft. (27 m) booms erects a tier of tower cells for each leg, then is jumped to the top of that tier and proceeds to erect the next tier. The tower legs are 90 ft. (27 m) center-to-center and 690 ft. (210 m) high. When the traveler completed the north tower (in background) it erected a Chicago boom on the west tower leg, which dismantled the creeper, erected tower-top bracing, and erected two small derricks (one shown) to service cable erection. Each tower contains 22,200 tons of steelwork.

FIGURE 1.23 Balanced-cantilever erection, Governor O.K. Allen railway/highway cantilever bridge, Baton Rouge, La., 1939. First use of long balanced-cantilever erection procedure in the U.S. On each pier 650 ft. (198 m) of steelwork, about 4000 tons, was balanced on the 40 ft. (12 m) base formed by a sloping falsework bent. The compression load at the top of the falsework bent was measured at frequent intervals and adjusted by positioning a counterweight car running at bottom-chord level. The main spans are 848-650-848 ft. (258-198-258 m); 650 ft. span shown. (Courtesy of Bethlehem Steel Corporation.)

FIGURE 1.24 Tower erection, second Tacoma Narrows Bridge, Washington, 1949. This bridge replaced first Tacoma Narrows bridge, which blew down in a 40 mph (18 m/sec) wind in 1940. Tower legs are 60 ft. (18 m) on centers and 462 ft. (141 m) high. Creeper traveler is shown erecting west tower, in background. On east tower, creeper erected a Chicago boom at top of south leg; this boom dismantled creeper, then erected tower-top bracing and a stifflegd errick, which proceeded to dismantle Chicago boom. Tower manhoist can be seen at second-from-topmost landing platform. Riveting cages are approaching top of tower. Note tower-base erection kneebraces, required to ensure tower stability in free-standing condition (see Figure 1.27).

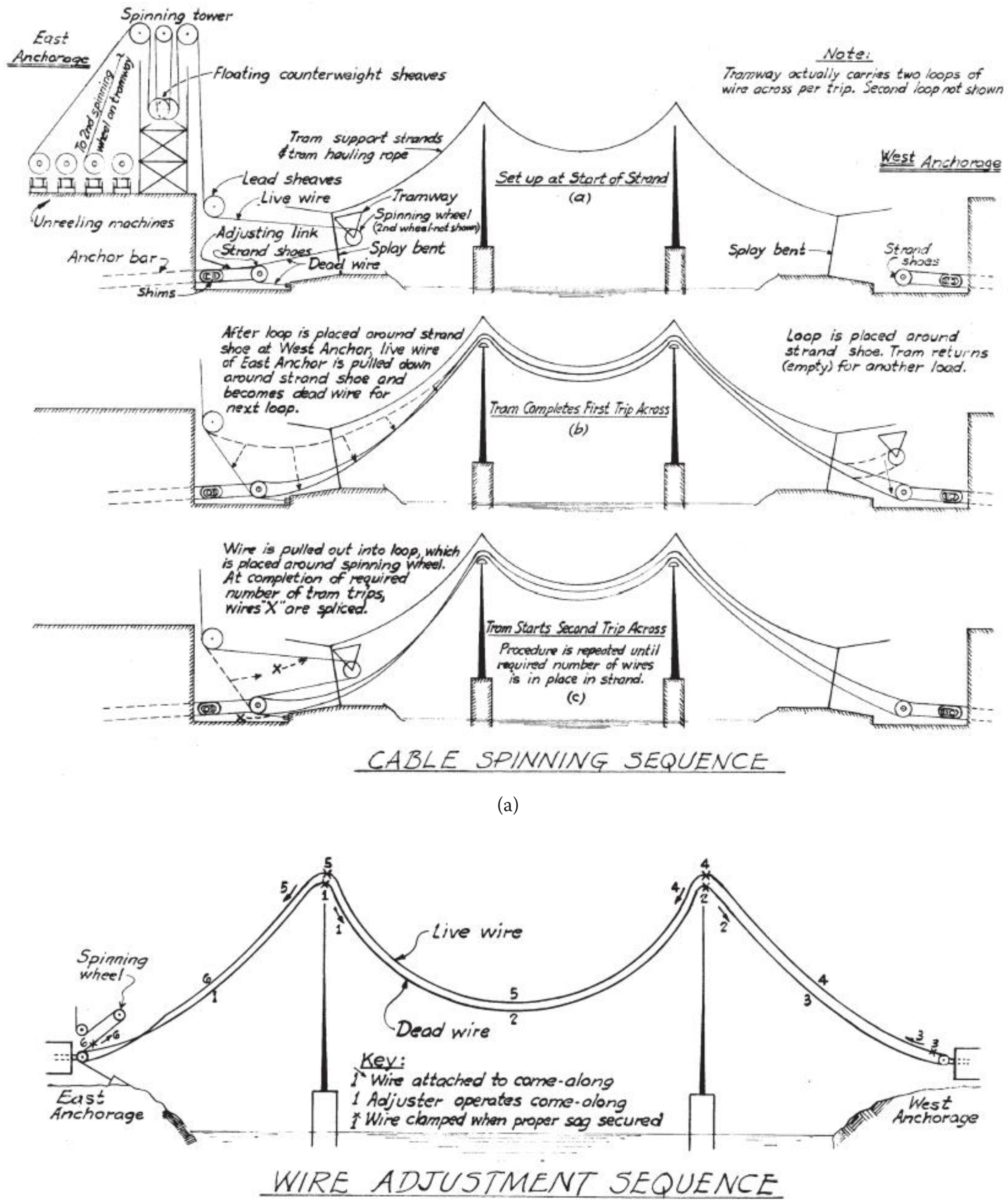


(a)



(b)

FIGURE 1.25 Aerial spinning of parallel-wire main cables, second Tacoma Narrows suspension bridge, Washington, 1949. Each main cable consists of 8702 parallel galvanized high-strength wires of 0.196 in. (4.98 mm) diameter, laid up as 19 strands of mostly 460 wires each. Following compaction the cable became a solid round mass of wires with a diameter of 20¼ in. (514 mm). (a) Tramway starts across from east anchorage carrying two wire loops. Three 460-wire strands have been spun, with two more under construction. Tramway spinning wheels pull wire loops across the three spans from east anchorage to west anchorage. Suspended footbridges provide access to cables. Spinning goes on 24 hours per day. (b) Tramway arrives at west anchorage. Wire loops shown in (a) are removed from spinning wheels and placed around strand shoes at west anchorage. This tramway then returns empty to east anchorage, while tramway for other "leg" of endless hauling rope brings two wire loops across for second strand that is under construction for this cable.



(a)

(b)

FIGURE 1.26 Cable-spinning procedure for constructing suspension bridge parallel-wire main cables, showing details of aerial spinning method for forming individual 5 mm wires into strands containing 400 to 500 wires. Each wire loop is erected as shown in (a) (refer to Figure 1.25), then adjusted to the correct sag as shown in (b). Each completed strand is banded with tape, then adjusted to the correct sag in each span. With all strands in place, they are compacted to form a solid round homogeneous mass of cable wires. The aerial spinning method was developed by John Roebling in the mid-19th century. (a) Erection of individual wire loops. (b) Adjustment of individual wire loops.

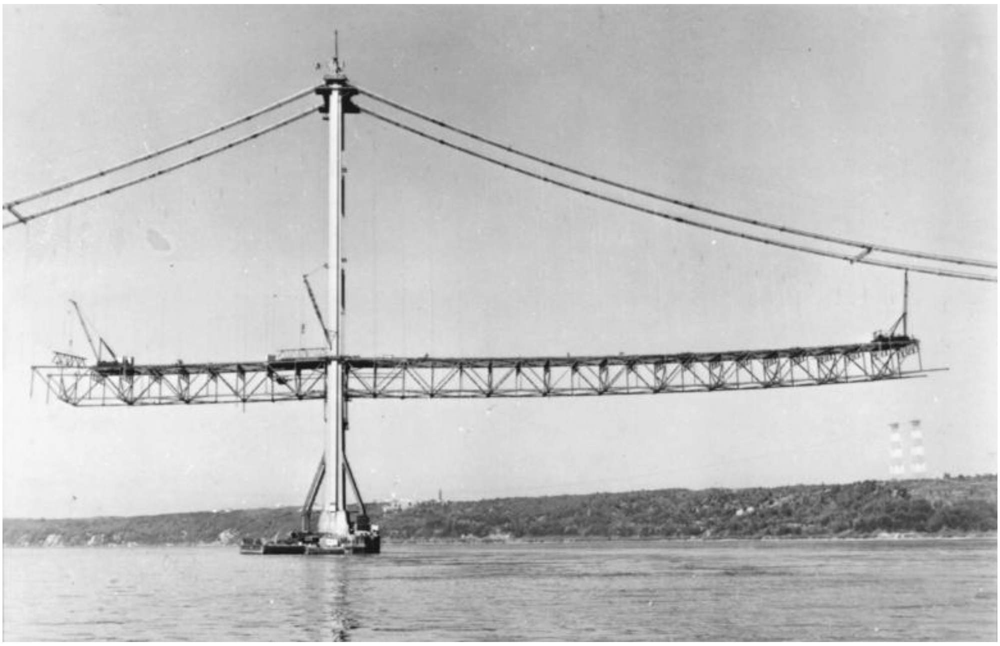
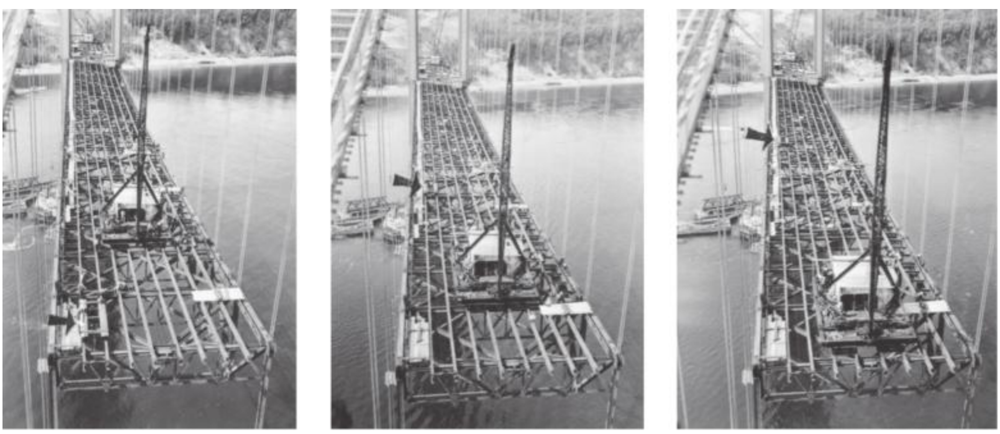


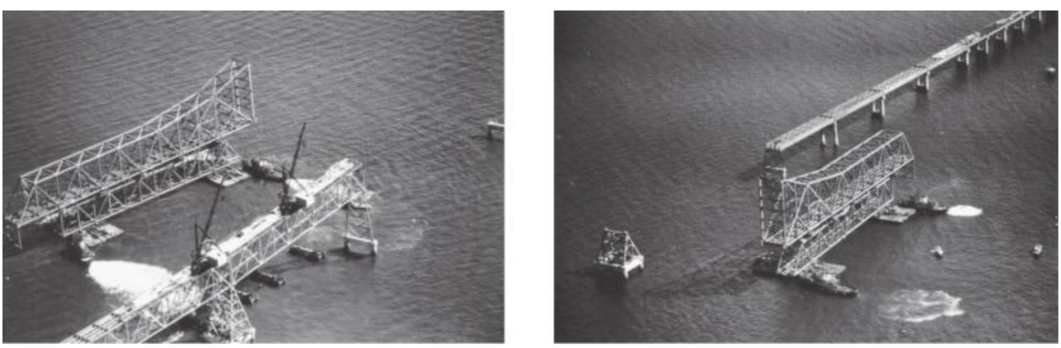
FIGURE 1.27 Erection of suspended deck steelwork, second Tacoma Narrows Bridge, Washington, 1950. Chicago boom on tower raises deck steelwork components to deck level, where they are transported to deck travelers by material cars. Each truss double panel is connected at top-chord level to previously erected trusses, and left open at bottom-chord level to permit temporary upward deck curvature, which results from partial loading condition of main suspension cables. Main span (at right) is 2800 ft. (853 m), and side spans are 1100 ft. (335 m). Stiffening trusses are 33 ft. (10 m) deep and 60 ft. (18 m) on centers. Tower-base kneebraces (see Figure 1.24) show clearly here.



(a) (b) (c)

FIGURE 1.28 Moving deck traveler forward, second Tacoma Narrows Bridge, Washington, 1950. Traveler pulling falls leadline passes around sheave beams at forward end of stringers, and is attached to front of material car (at left). Material car is pulled back toward tower, advancing traveler two panels to its new position at end of deck steelwork. Arrows show successive positions of material car. (a) Traveler at start of move, (b) traveler advanced one panel, and (c) traveler at end of move.

FIGURE 1.29 Erecting closing girder sections of Passaic River Bridge, New Jersey Turnpike, 1951. Huge double-boom travelers, each weighing 270 tons, erect closing plate girders of the 375 ft. (114 m) main span. Closing girders are 14 ft. (4.3 m) deep and 115 ft. (35 m) long and weigh 146 tons each. Sidewise entry was required (as shown) because of long projecting splice material. Longitudinal motion was provided at one pier, where girders were jacked to effect closure. Closing girders were laterally stable without floor steel fill-in, such that derrick falls could be released immediately. (Courtesy of Bethlehem Steel Corporation.)



(a) (b)

FIGURE 1.30 Floating-in erection of a truss span, first Chesapeake Bay Bridge, Maryland, 1951. Erected 300 ft. (91 m) deck-truss spans form erection dock, providing a work platform for two derrick travelers. A permanent deck-truss span serves as a falsework truss supported on barges and is shown carrying the 470 . (143 m) anchor arm of the through-cantilever truss. This span is floated to its permanent position, then landed onto its piers by ballasting the barges. (a) Float leaves erection dock, and (b) float arrives at permanent position. (Courtesy of Bethlehem Steel Corporation.)

FIGURE 1.31 Floating-in erection of a truss span, first Chesapeake Bay Bridge, Maryland, 1952. A 480 ft. (146 m) truss span, weighting 850 tons, supported on falsework consisting of a permanent deck-truss span along with temporary members, is being floated-in for landing onto its piers. Suspension bridge cables are under construction in background. (Courtesy of Bethlehem Steel Corporation.)

FIGURE 1.32 Erection of a truss span by hoisting, first Chesapeake Bay Bridge, Maryland, 1952. A 360 ft. (110 m) truss span is floated into position on barges and picked clear using four sets of lifting falls. Suspension bridge deck is under construction at right. (Courtesy of Bethlehem Steel Corporation.)

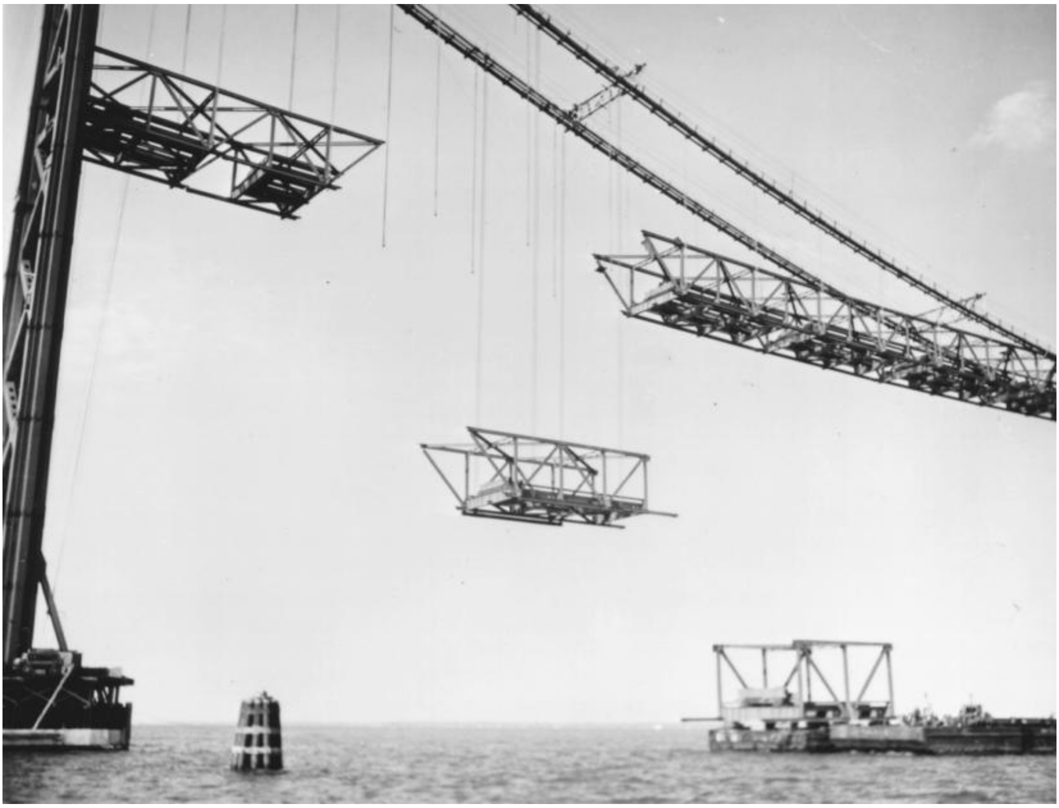


FIGURE 1.33 Erection of suspension bridge deck structure, first Chesapeake Bay Bridge, Maryland, 1952. A typical four-panel through-truss deck section, weighing 99 tons, has been picked from the barge and is being raised into position using four sets of lifting falls attached to main suspension cables. Closing deck section is on barge, ready to go up next. (Courtesy of Bethlehem Steel Corporation.)

FIGURE 1.34 Greater New Orleans cantilever bridge, Louisiana, 1957. Tall double-boom deck travelers started at ends of main bridge and erected anchor spans on falsework, then the 1575 ft. (480 m) main span by cantilevering to midspan. (Courtesy of Bethlehem Steel Corporation.)

FIGURE 1.35 Tower erection, second Delaware Memorial Bridge, Wilmington, Del., 1966. Tower erection traveler has reached topmost erecting position and swings into place 23-ton closing top-strut section. Tower legs were jacked apart about 2 in. (50 mm) to provide entering clearance. Traveler jumping beams are in topmost working position, above cable saddles. Tower steelwork is about 418 ft. (127 m) high. Cable anchorage pier is under construction at right. First Delaware Memorial Bridge (1951) is at left. The main span of both bridges is 2150 ft. (655 m). (Courtesy of Bethlehem Steel Corporation.)

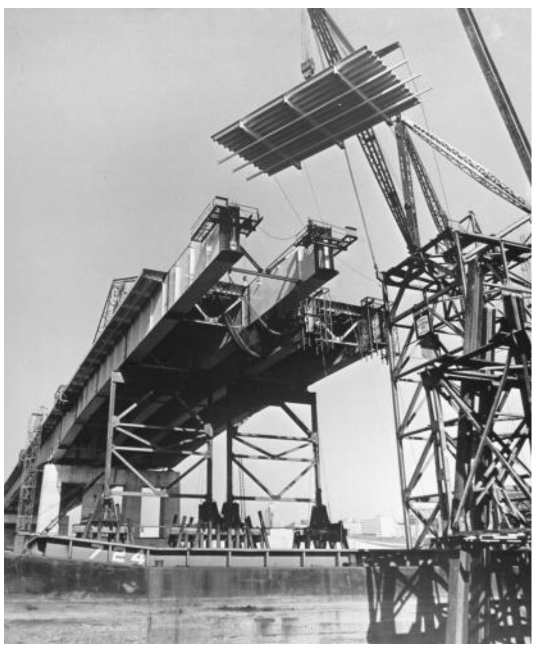


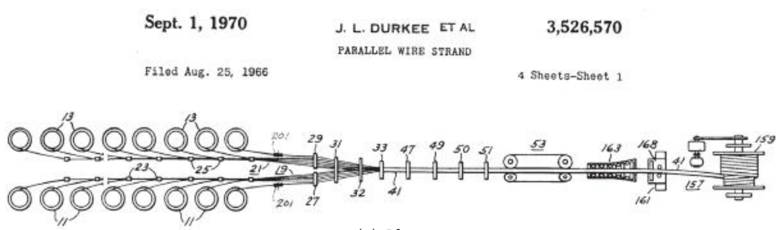
FIGURE 1.36 Erecting orthotropic-plate decking panel, Poplar Street Bridge, St. Louis, Mo., 1967. A five-span, 2165 ft. (660 m) continuous box-girder bridge, main span 600 ft. (183 m). Projecting box ribs are 5-1/2 × 17 ft. (1.7 × 5.2 m) in cross-section, and decking section is 27 × 50 ft. (8.2 × 15.2 m). Decking sections were field welded, while all other connections were field bolted. Box girders are cantilevered to falsework bents using overhead "positioning travelers" (triangular structure just visible above deck at left) for intermediate support. (Courtesy of Bethlehem Steel Corporation.)

(a)

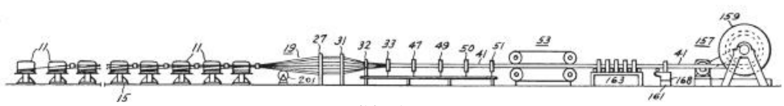
(b)

FIGURE 1.37 Erection of parallel-wire-stand (PWS) cables, Newport Bridge suspension spans, Narragansett Bay, R.I., 1968. Bridge engineering history was made at Newport with the development and application of shop-fabricated parallel-wire socketed strands for suspension bridge cables. Each Newport cable was formed of seventy-six 61-wire PWS, each 4512 ft. (1375 m) long and weighing 15 tons. Individual wires are 0.202 in. (5.13 mm) in diameter and are zinc coated. Parallel-wire cables can be constructed of PWS faster and at lower cost than by traditional air spinning of individual wires (see Figures 1.25 and 1.26). (a) Aerial tramway tows PWS from west anchorage up side span, then on across other spans to east anchorage. Strands are about 1¾ in. (44 mm) in diameter. (b) Cable formers maintain strand alignment in cables prior to cable compaction. Each finished cable is about 15¼ in. (387 mm) in diameter. (Courtesy of Bethlehem Steel Corporation.)

FIGURE 1.38 Pipe-type anchorage for parallel-wire-strand (PWS) cables, Newport Bridge suspension spans, Nar ragansett Bay, R.I., 1967. Pipe anchorages shown will be embedded in anchorage concrete. Socketed end of each PWS is pulled down its pipe from upper end, then seated and shim-adjusted against heavy bearing plate at lower end. Pipe-type anchorage is much simpler and less costly than standard anchor-bar type used with aerial-spun parallel-wire cables (see Figure 1.25B). (Courtesy of Bethlehem Steel Corporation.)



(a) Plan



(b) Elevation

FIGURE 1.39 Manufacturing facility for production of shop-fabricated parallel-wire strands (PWS). Prior to 1966, parallel-wire suspension bridge cables had to be constructed wire-by-wire in the field using aerial spinning procedure developed by John Roebling in the mid-19th century (refer to Figures 1.25 and 1.26). In the early 1960s a major U.S. steelwork contractor originated and developed a procedure for manufacturing and reeling parallel-wire strands, as shown in these patent drawings. A PWS can contain up to 127 wires (see Figures 1.45 and 1.46). (a) Plan view of PWS facility. Turntables 11 contain “left-hand” coils of wire and turntables 13 contain “right-hand” coils, such that wire cast is balanced in the formed strand. Fairleads 23 and 25 guide the wires into half-layplates 27 and 29, followed by full layplates 31 and 32 whose guide holes delineate the hexagonal shape of final strand 41. (b) Elevation view of PWS facility. Hexagonal die 33 contains six spring-actuated rollers that form the wires into regular-hexagon shape; and similar roller dies 47, 49, 50, and 51 maintain the wires in this shape as PWS 41 is pulled along by hexagonal dynamic clamp 53. PWS is bound manually with plastic tape at about 3 ft. (1 m) intervals as it passes along between roller dies. PWS passes across roller table 163, then across traverse carriage 168, which is operated by traverse mechanism 161 to direct the PWS properly onto reel 159. Finally, reeled PWS is moved off-line forsocketing. Note that wire measuring wheels 201 can be installed and used for control of strand length.

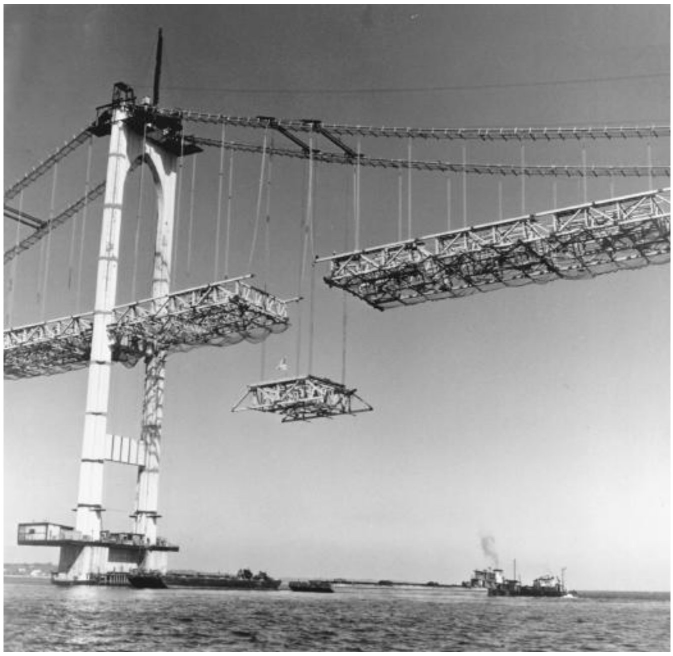
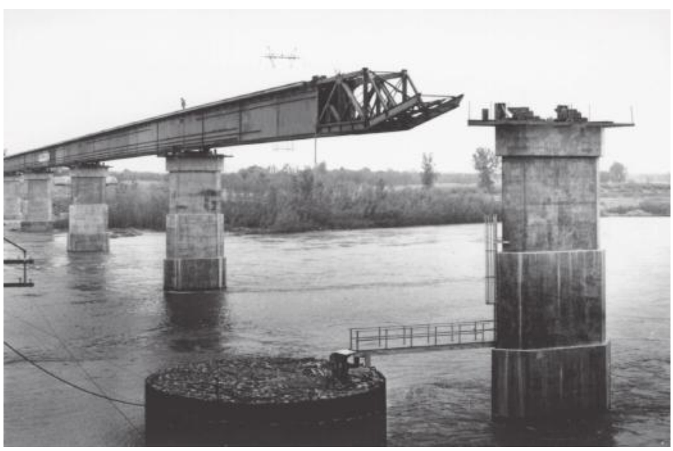


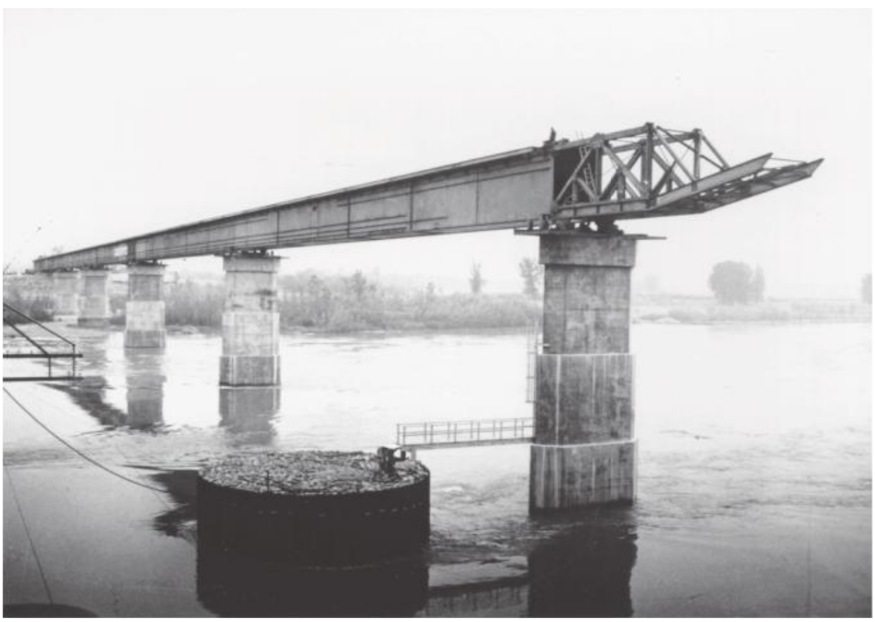
FIGURE 1.40 Suspended deck steelwork erection, Newport Bridge suspension spans, Narragansett Bay, R.I., 1968. Closing mainspan deck section is being raised into position by two cable travelers, each made up of a pair of 36 in. (0.91 m) wide-flange rolled beams that ride cables on wooden wheels. Closing section is 40-1/2 ft. (12 m) long at top-chord level, 66 ft. (20 m) wide and 16 ft. (5 m) deep, and weighs about 140 tons. (Courtesy of Bethlehem Steel Corporation.)



(a)

FIGURE 1.41 Erection of Kansas City Southern Railway box-girder bridge, near Redland, Okla., by "launching," 1970. This nine-span continuous box-girder bridge is 2110 ft. (643 m) long, with a main span of 330 ft. (101 m). Box cross section is 11 × 14.9 ft. (3.35 × 4.54 m). Girders were launched in two "trains," one from north end and one from south end. A "launching nose" was used to carry leading end of each girder train up onto skidway supports as train was pushed out onto successive piers. Closure was accomplished at center of main span. (a) Leading end of north girder train moves across 250 ft. (76 m) span 4, approaching pier 5. Main span, 330 ft. (101 mm), is to right of pier 5.

(b)



(c)

FIGURE 1.41 (*Continued*) Erection of Kansas City Southern Railway box-girder bridge, near Redland, Okla., by "launching," 1970. This nine-span continuous box-girder bridge is 2110 ft. (643 m) long, with a main span of 330 ft. (101 m). Box cross section is 11 × 14.9 ft. (3.35 × 4.54 m). Girders were launched in two "trains," one from north end and one from south end. A "launching nose" was used to carry leading end of each girder train up onto skidway supports as train was pushed out onto successive piers. Closure was accomplished at center of main span. (b) Launching nose rides up onto pier 5 skidway units, removing girder-train leading-end sag. (c) Leading end of north girder train is now supported on pier 5. (Courtesy of Bethlehem Steel Corporation.)

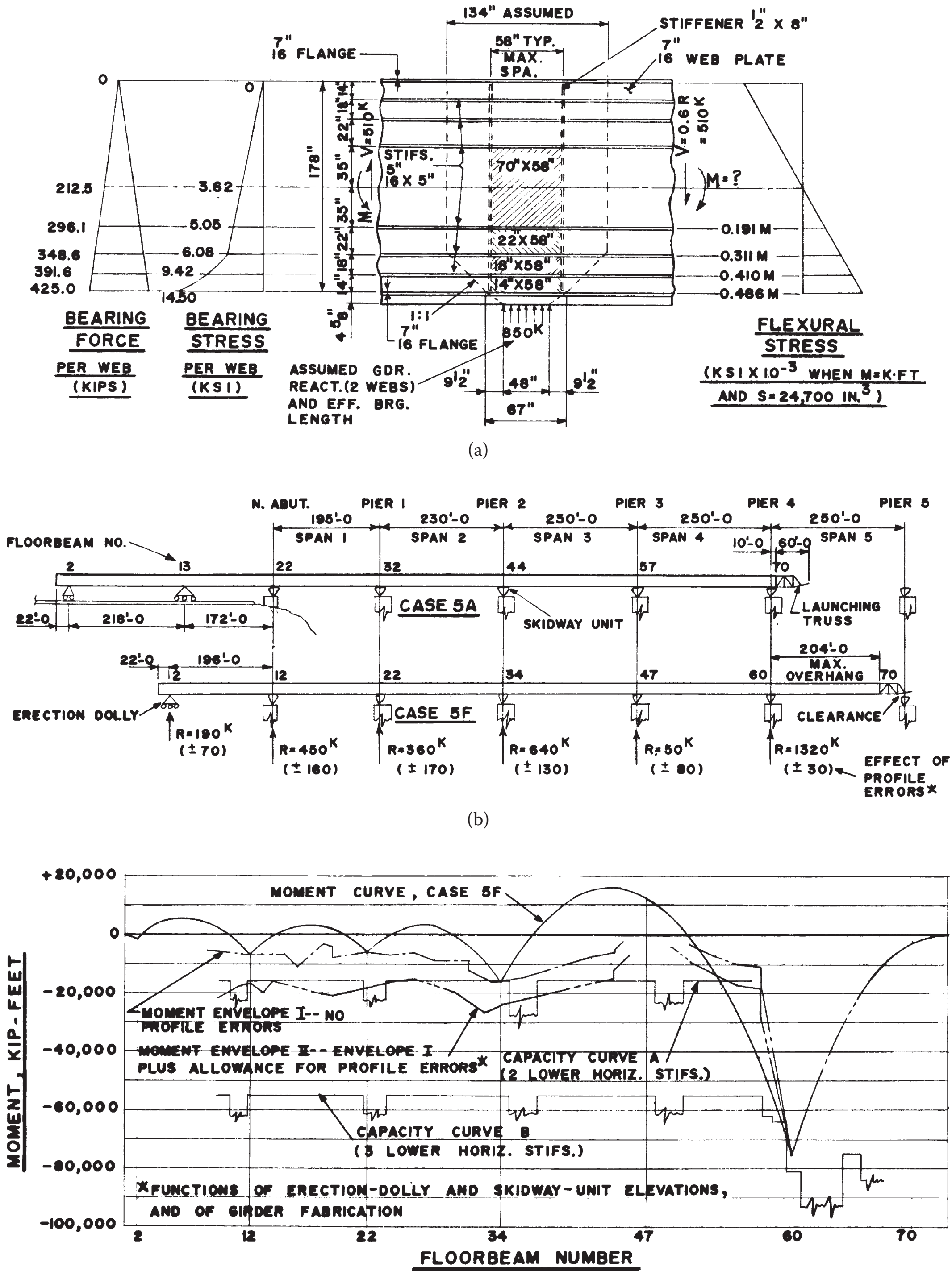


FIGURE 1.42 Erection strengthening to withstand launching, Kansas City Southern Railway box-girder bridge, near Redland, Okla. (see Figure 1.41). (a) Typical assumed erection loading of box-girder web panels in combined moment, shear, and transverse compression. (b) Launch of north girder train from pier 4 to pier 5. (c) Negative-moment envelopes occurring simultaneously with reaction, for launch of north girder train to pier 5.

FIGURE 1.43 Erection of west arch span of twin-arch Hernando de Soto Bridge, Memphis, Tenn., 1972. The two 900 ft. (274 m) continuous-truss tied-arch spans were erected by a high-tower derrick boat incorporating a pair of barges. West-arch steelwork (shown) was cantilevered to midspan over two pile-supported falsework bents. Projecting east-arch steelwork (at right) was then cantilevered to midspan (without falsework) and closed with falsework-supported other half-arch. (Courtesy of Bethlehem Steel Corporation.)

FIGURE 1.44 Closure of east side span, Commodore John Barry cantilever truss bridge, Chester, Pa., 1973. High-tower derrick boat (in background) started erection of trusses at both main piers, supported on falsework; then erected top-chord travelers for main and side spans. Sidespan traveler carried steelwork erection to closure, as shown, and falsework bent was then removed. East-mainspan traveler then cantilevered steelwork (without false-work) to midspan, concurrently with cantilever erection by west-half mainspan traveler, and trusses were closed at midspan. Commodore Barry has 1644 ft. (501 m) main span, longest cantilever span in the U.S., and 822 ft. (251 m) side spans. (Courtesy of Bethlehem Steel Corporation.)

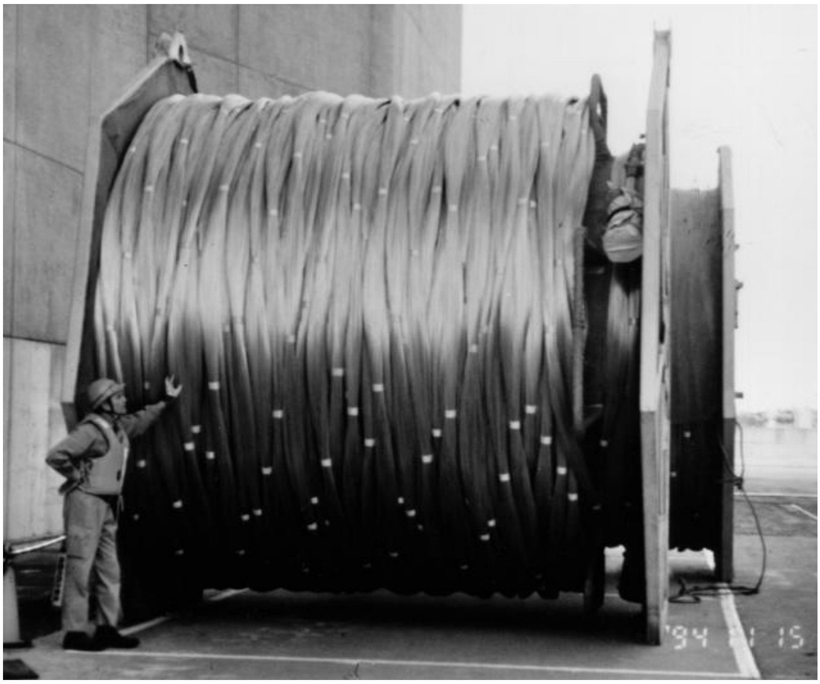


FIGURE 1.45 Reel of parallel-wire strand (PWS), Akashi Kaikyo suspension bridge, Kobe, Japan, 1994. Each socketed PWS is made up of 127 0.206 in. (5.23 mm) wires, is 13,360 ft. (4073 m) long, and weighs 96 tons. Plastic-tape bindings secure the strand wires at 1 m intervals. Sockets can be seen on right side of reel. These PWS are the longest and heaviest ever manufactured. (Courtesy of Nippon Steel—Kobe Steel.)

FIGURE 1.46 Parallel-wire-strand main cable, Akashi Kaikyo suspension bridge, Kobe, Japan, 1994. Main span is 6532 ft. (1991 m), by far the world's longest. PWS at right is being towed across spans, supported on rollers. Completed cable is made up of 290 PWS, making a total of 36,830 wires, and has diameter of 44.2 in. (1122 mm) following compaction—largest bridge cables built to date. Each 127-wire PWS is about 2-3/8 in. (60 mm) in diameter. (Courtesy of Nippon Steel—Kobe Steel.)

FIGURE 1.47 Artist's rendering of proposed Messina Strait suspension bridge connecting Sicily with mainland Italy. The Messina Strait crossing has been under discussion since about 1870, under investigation since about 1955, and under active design since about 1975. The first realistic proposals for a crossing were made in 1969 in response to an international competition sponsored by the Italian government. There were 158 submissions—eight American, three British, three French, one German, one Swedish, and the remaining Italian. Forty of the submissions showed a single-span or multi-span suspension bridge. The enormous bridge shown has a single span of 10,827 ft. (3300 m) and towers 1250 ft. (380 m) high. The bridge construction problems for such a span would be tremendously challenging. (Courtesy of Stretto di Messina, S.p.A.)

References

Copp, J.I., de Vries, K., Jameson, W.H. and Jones, J. 1945. Rainbow Arch Bridge over Niagara George: A Symposium: Fabrication and Erection Control, *Transactions*, ASCE, 110(1), 50–81.

Durkee, J.L. 1945. Rainbow Arch Bridge over Niagara George: A Symposium: Erection of Steel Superstructure, *Transactions,* ASCE, 110(1), 82–119.

Durkee, J.L. 1966. Advancements in Suspension Bridge Cable Construction, *Proceedings, International Symposium on Suspension Bridges*, Laboratorio Nacional de Engenharia, Lisbon, Portugal.

Durkee, J.L. 1977. Needed: U.S. Standard Conditions for Contracting, *J. Constr. Div,* 103(2), 191–200.

Durkee, J.L. and Thomaides, S.S. 1977. Erection Strength Adequacy of Long Truss Cantilevers, *J. Struct. Div,* ASCE, 103(1), 225–262.

Feidler, L.L., Jr. 1962. Erection of the Lewiston-Queenston Bridge, *Civil Engrg*, ASCE, 83(11), 50–53.

ICE. 2003. *ICE Conditions of Contract*, 7th Edition, Thomas Telford Publishing, London, UK.

Pugsley, A.G. 1968. The Safety of Bridges, *The Structural Engineer*, UK, July, 197–201.

Bibiliography

Durkee, J.L. 1972. Railway Box-Girder Bridge Erected by Launching, *J. Struct. Div*, ASCE, 98(7), 1443–1463.

Durkee, J.L. 1982. Bridge Structural Innovation: Firsthand Report, *J. Prof. Act.* 108(3), 190–201.

Freudenthal, A.M. Ed. 1972. The Engineering Climatology of Structural Accidents, *Proceedings of the International Conference on Structural Safety and Reliability*, Pergamon Press, Elmsford, NY.

Holgate, H. Herry, J.G.G., and Galbraith, J. 1908. *Royal Commission Quebec Bridge Inquiry Report*, Session Paper No. 154, Vol. I and II, S.E. Dawson, Ottawa, Canada.

Leto, I.V. 1994. Preliminary Design of the Messina Strait Bridge, *Proc. Inst. Civil Engrs*, UK, 102(3), 122–129.

Merrison A.W. 1973. *Inquiry into the Basis of Design and Method of Erection of Steel-Box Girder Bridges Report of the Committee*, HMSO, London, UK.

Petroski, H. 1993. Predicting Disaster, *American Scientist*, 81(2), 110–113.

Ratay, R.T., Ed. 1996. *Handbook of Temporary Structures in Construction,* 2nd ed., McGraw-Hill, New York, NY.

Schneider, C.C. 1905. The Evolution of the Practice of American Bridge Building, *Transactions, ASCE,* 54(1), 213–234.

Sibly, P.G. and Walker, A.C. 1977. Structural Accidents and Their Causes, *Proc. Inst. Civil Engrs,* UK, 62(2), 191–208.

Smith, D.W. 1976. Bridge Failures, *Proc. Inst. Civil Engrs,* UK, 61(3), 367–382.

2

Steel Bridge Fabrication

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2.1 Introduction

This chapter addresses key challenges and processes for steel girder bridge fabrication to help the engineer facilitate projects that achieve goals for cost, quality, and schedule. For the fabricator, the key to achieving these goals is execution of the project according to plan. Hence designs accomplished with an understanding of fabrication values have a better chance of success.

2.2 Standardization

The fabricator commits to a price and schedule when they win a job. Both are based on how they intend to use their people and equipment to accomplish the job. The fabricator's probability of success on the project depends on how well they understand the plans and specifications at bid time, and also on the support they get from other members of the project team during project execution for such things as responses to requests for information (RFIs), shop drawing review and approval, fabrication practice restrictions and tolerances, and inspection. Therefore, a successful project is use of standard practices for design, specifications, and contract practices (such as answering RFIs and shop drawing review). Figure 2.1 illustrates relationship between the standardization and success of project.

Conversely, lack of standardization can lead to costly problems:

- Nonstandard details and design presentation that cause confusion
- Lack of RFI answers that cause delays
- Shop drawing review and approval delays
- Unusual fabrication process restrictions and tolerances that add time and cost
- Unexpected inspection practices and demands that add time and cost

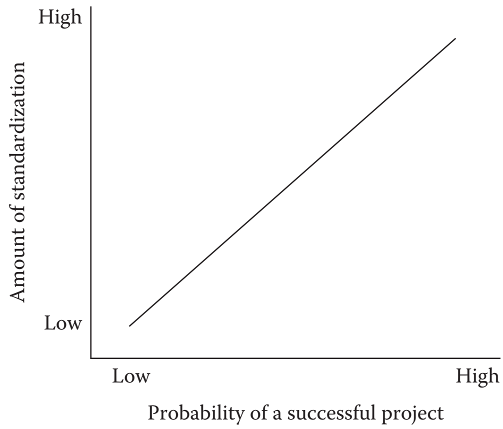

FIGURE 2.1 The probability of success on a fabrication practice is greater with standardization.

In sum, use of standard practices in design and construction is essential to achieving a project that is on schedule and achieves the best possible economy and quality.

2.3 Prefabrication

Fabrication is usually envisaged as shop activities such as cutting and welding steel, but much of the project work is actually in prefabrication, before the first cutting torches are lit. The key activities in prefabrication are described in Sections 2.3.1 through 2.3.6.

2.3.1 Fabrication Schedule

It may take only a few days of actual fabrication to produce bridge girders, but as reflected in the schedule shown in Figure 2.2, achievement of a successful project usually takes many weeks of preparation and relies upon support from other project members.

Preferably, fabrication will not begin until shop drawings are approved. Not only do most bridge owners require this, but also fabricators prefer not to start work under the risk that project requirements could change during the shop drawing review and approval process. Unfortunately, if drawing approval is delayed, the fabricator may be forced to begin work to meet a delivery commitment. When a fabricator wins a job and commits to a delivery, they include time for shop drawing review and approval in their plan, but they cannot control this time and must rely on cooperation from the owner and design for review in a timely manner. To help, engineers should review the shop drawings expeditiously and approve drawings "as-noted" to the extent possible for small corrections.

The fabricator must also rely on support to prepare shop drawings. Many projects require design clarifications, which the fabricator (or their detailer) will seek through RFIs. Projects may also require field measurements from the general contractor (GC) or clarification of utility information.

In addition to approved drawings, the fabricator must have materials to start a job. Smaller plates and shapes are readily available on the market through service centers, but large plates for components such as girder flanges and webs must be special ordered. The lead time for such materials varies—thinner sections of traditional grades such as ASTM A709 Grade 36, A709 Grade 50, and A709 Grade 50W may be available in as few as eight weeks, but high performance steel takes longer, particularly thicker sections that must be quenched and tempered. The fabricator will be mindful of current lead times for the various materials to be used on the project and will order the material accordingly. For an aggressive delivery, the fabricator must order material as soon as possible after winning a project, so one of their first steps is to produce a bill of materials (BOM) for main member materials. Therefore, a design change that alters main member material can put a project schedule in serious jeopardy.

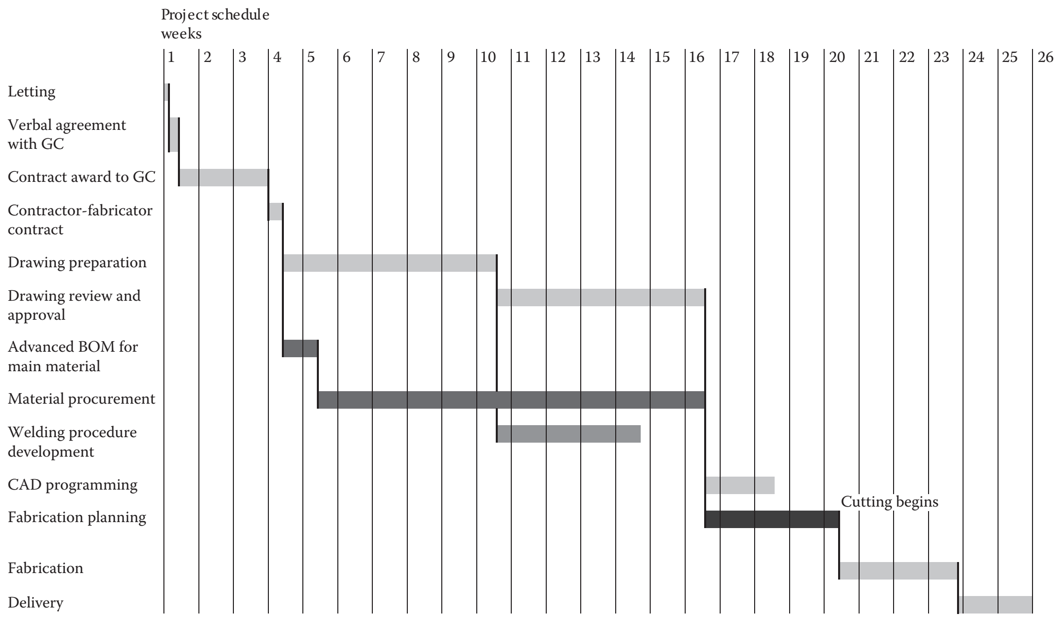

FIGURE 2.2 This idealized steel bridge project schedule shows that drawings and materials must be ready to start fabrication.

2.3.2 Shop Drawing Preparation

Though the role of a shop drawing has changed with the advance of technology, the shop drawings still define and clearly communicate to everyone in the shop exactly how the job is to be accomplished. Since shop drawings are often revised along the course of the project, most fabricators have strict rules in place to ensure the proper shop drawing is in the shop circulation and is in control. Often drawings go through multiple revisions. As drawings are revised, it is imperative that only the current and correct drawing is in circulation on the shop floor.

Before the advent of computer numerically controlled (CNC) equipment, all fabrication operations were accomplished by hand: shop workers would read dimensions from the shop drawings, measure the steel with steel tapes, lay out markings with soap stone, and use hand-stamped punch marks to locate future bolt holes. These techniques are still used today, but many former hand fabrication steps are now performed with computer automation. Therefore, the role of shop drawings is changing because machines need code and not drawings. Certainly, machine programmers read drawings to produce code, but much of the code is created automatically as the drawings are made.

Shop workers must be able to read and understand the shop drawings without confusion. To achieve this, fabricators use consistent practices to present information on shop drawings, from standard nomenclature for welding, drilling, and finishing operations to standard drawing names, drawing numbers, and piece marks. For all fabrication information presentation, consistent presentation facilitates understanding, and a change in practices is an opportunity for confusion, error, cost overruns, and delays.

Shop drawings are prepared by the fabricator's in-house staff or by a subcontract bridge detailer. Though there are many structural steel detailers in the marketplace, only a few have the bridge knowledge and skills to detail steel bridges. Steel bridge detailers have special bridge knowledge, such as how to translate the bridge deflections and bridge geometrics, including bearing elevations, steel member geometry, bridge deck geometry, and bridge layout into the dimensions needed to fabricate steel girders. Detailers use customized spreadsheets or their own software to work out the bridge coordinate geometry; such automation is not presently available on the market, so it is home grown.

To facilitate ready and accurate understanding of the drawings, the detailer uses a standard approach to accomplish the drawings, in alignment with shop preferences. For example, the fabricator can use any means of labeling elements and numbering drawings that they choose, but common protocols have become customary. These are described in American Association of State Highway and Transportation Officials (AASHTO)/National Steel Bridge Alliance (NSBA) Steel Bridge Collaboration Standard G1.3, *Shop Detail Drawing Presentation Guidelines* (AASHTO/NSBA 2002). This standard is available from the NSBA or through the AASHTO bookstore.

Almost without exception the fabricator will have questions that must be answered to complete the shop drawings. Fundamentally, because the engineer does not produce shop drawings, they do not understand everything that the detailer needs to accomplish the job. Questions from the detailer are normal and to be expected. To keep a project on schedule, the engineer must answer RFIs expeditiously, thoroughly, and accurately.

Direct lines of communication offer superior processing of detailing information. It is recognized that, officially, the fabricator works for the GC, who in turn has a contract with the owner, and the engineer who will answer RFIs and review shop drawings works for the owner. The owner or GC may insist that all communication between the fabrication and the engineer strictly follow official lines of communication to protect their interests, but this complicates communications, adding time and increasing the possibility of confusion, churn, and error. It is much more effective to establish an arrangement where communications can be direct, including the possibility of verbal discussion for complicated matters. The best practice is to allow direct, written communication between the fabricator and engineer, with copy of the communication to the GC, the owner, and anyone concerned. With email, this is very readily accomplished. If the fabricator must only write through the GC and owner to the engineer, allow the engineer to be copied on communications to the GC. This is less effective than direct communication between the fabricator and engineer, but at least this will allow the engineer to begin considering their response.

2.3.3 Shop Drawing Review and Approval

Shop drawing review and approval is on the fabricator's critical path and should be handled as expeditiously as possible to help keep the project on schedule.

Shop drawing review and approval helps ensure the fabricator's intent is consistent with the engineer's intent and it helps avoid errors. But there are costs associated with this process, and the process can be detrimental to the project if it causes delays.

The owner should establish the responsibility for shop drawing review and approval at the beginning of the project. Often this process is handled by the design engineer; if this is the case and the designer is a consultant, the owner should provide clear direction and expectations to the reviewer and ensure that there is adequate time in the contract for the review.

The customary intent of the process is well defined in the AASHTO/NSBA Steel Bridge Collaboration G1.1, *Shop Detail Drawing Review and Approval Guidelines* (AASHTO/NSBA 2000), which includes these key details:

- 1.6: "Approval" is a verification that the drawings appear to be consistent with the contract drawings; approval does not relieve the fabricator of responsibility for the correctness of the drawings.
- 2.1: The fabricator is responsible for producing correct drawings.
- 2.2: For most structures, drawings should be returned within three to four weeks; complex structures may take longer; partial submittals should be allowed.
- 2.3: All parties involved should work together as closely as possible to facilitate approval. If the engineer cannot understand something in the drawings or finds what he/she believes to be a substantial error, he/she should call the fabricator to help resolve the issue. The fabricator may be able to clear things up; or, if there is a significant error, the fabricator can get busy making the correction.

- Section 4: This section provides an approval checklist.
- 5.2: The engineer should strive to use "approved" or "approved as noted" for expediency; "rejected" or "revise and resubmit" should be applied only for serious problems. Further, if there are errors that justify a rejection, reject only the drawings in question and not the entire package.

As reflected in the hypothetical schedule in Figure 2.2, shop drawing approval is on the fabricator's critical path. When scheduling the project, the fabricator must make assumptions about how long shop drawing approval will take, and these will be based on their experience with the owner and the customary practices that are reflected in G1.1. This heightens the importance of the engineer working as closely as possible with the fabricator to complete the approval process.

2.3.4 Materials

Fabricators custom-order large plates for steel bridges, such as those for girder webs and flanges, directly from mills and purchase smaller plates from steel service centers, which typically carry only thinner plates and lengths up to 20 ft. For most projects, steel bridge fabricators will use a mix of custom-ordered large plates for web and flanges and smaller plates purchased from a service center for items such as stiffeners and gusset plates. Similarly, service centers carry smaller structural shapes such as angles and tees for crossframes, but large rolled beams usually must also be custom-ordered (Figure 2.3).

Generally, it takes many weeks to get steel bridge plates from mills. Lead times vary with thickness and grade, and depend on market conditions. Typically, grade 50 and 50W steels are available in 12–16 weeks; thinner high performance steel (HPS) plates can take a few weeks longer, and thicker HPS plates take longer still. Thinner HPS plates can be produced by thermal-mechanical control processing, but in the United States thicker plates are produced by quench-and-tempering processing, and fewer mills have this capability. Delivery times for large rolled shapes are similar; large shapes are not kept in stocked but are rolled on demand, and mills may roll such shapes two or three times a year. The lead times suggested above are typical but change with demand, so it will be a good idea to check with an industry representative when planning an urgent project.

Given that lead times run several weeks, materials are frequently on the critical path of a project schedule, particularly on time-sensitive projects. Fabricators order material as soon as possible to help ensure that the project will be on schedule. Once the basic bridge geometry is established during shop

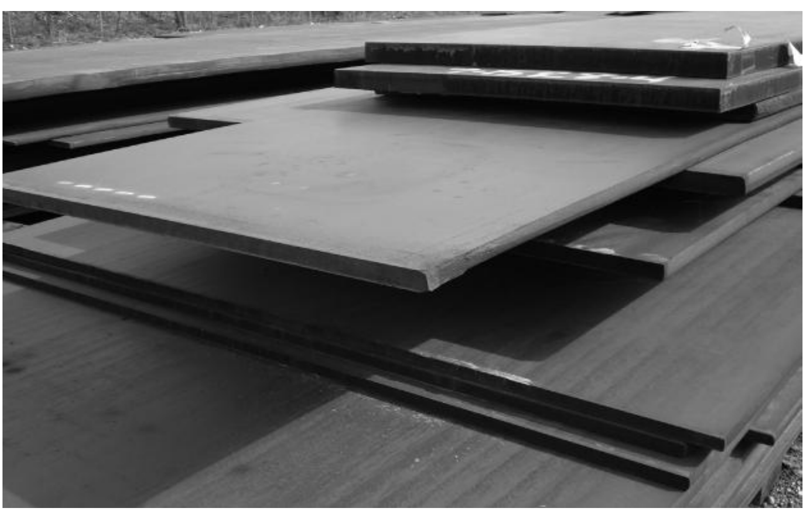

FIGURE 2.3 Most steel bridges start out as steel plate, which is cut, fitted, and welded into final bridge components.

drawing development, the detailer will produce an advance BOM to produce the order. Typically, materials must be ordered well before shop drawings are submitted for review, so it is important that the basic geometry is correct. If the fabricator finds errors in the bridge geometry, they will address these with the designer through the RFI process. Materials cannot be ordered until such RFIs are answered, so, once again, it is important to answer such RFIs as expeditiously as possible. Further, downstream shop drawing development also depends on the basic geometry information, so outstanding questions can bring detailing to a halt until the questions are resolved.

2.3.5 Fabrication Planning

Fabrication planning relies on approved shop drawings, but planning actually begins during estimating, when a fabricator first begins to consider how he/she will accomplish the project with his/her equipment and people. The fabricator makes basic assumptions during estimating and completes plans in earnest once shop drawings are ready.

Planning involves making dozens of decisions about how the work will be accomplished. How will welding be accomplished? How will drilling be accomplished? If the fabricator has multiple work areas, which one(s) will be used for the project? How will the work flow through the plant? Which equipment will be used for drilling and cutting, and will it be available? These and many other questions must be answered before the project starts to keep the work moving smoothly once it is on the shop floor (Figure 2.4).

2.3.6 CAD/CAM Programming

Concurrent with final planning, the fabricator will produce computer-aided design and computer-aided manufacturing (CAD/CAM) programs for CNC equipment once drawings are approved. Programmers create part models from drawing information and use the models to generate CNC code and tool path information. Though the models are based on shop drawing dimensions, CAD/CAM programmers adjust the models for manufacturing. For example, the fabricator may add extra length to a plate to account for weld shrinkage.

FIGURE 2.4 During project planning, a fabricator may make scale models of the components to be fabricated to help him work out the best fabrication approach.

2.4 Fabrication

Once shop drawings are complete and approved, programs are ready, welding procedures are in place, and material has arrived, actual fabrication processes may begin.

2.4.1 Cutting

Plates and shapes are cut down from the ordered size to the size needed in the shop. Thermal cutting is used most frequently, but saws are often used to cut large shapes, and some fabricators may still use a shear for smaller plate cutting.

Plates are cut on tables using a thermal process. Oxygen cutting and plasma cutting are the predominant methods used. Other methods such as laser and water-jet can effectively cut steel, but both these methods are limited to smaller thicknesses. Further, as of this writing, it is not known if holes cut with such methods have been tested for suitability in bridge fabrication.

Oxygen cutting, or oxy-acetylene cutting, is the most common cutting method used for steel fabrication due to its versatility. Oxy-acetylene can cut steel over one foot thick, so it readily handles all common bridge plate thicknesses (Figure 2.5).

Plasma cutting is generally considered to be superior to oxy-acetylene cutting because it is faster and generally provides a cleaner cut. However, plasma can only readily handle plates up to 2 or 2.5 in. thick (depending on the equipment) without slowing down below practical speeds, so fabricators who use plasma will also cut some components with oxy-acetylene. Plasma cutting, as shown in Figure 2.6, is

FIGURE 2.5 Oxy-acetylene cutting is shown being used to prepare a plate for welding.

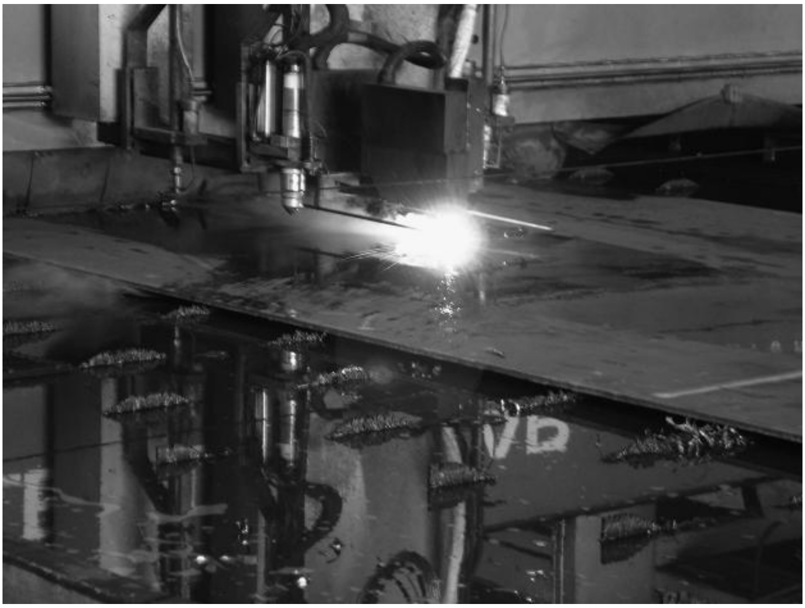

FIGURE 2.6 Plasma cutting.

popular for cutting steel up to about 2 in. thick. Typically plasma is used over water to reduce noise and smoke in the shop.

The NCHRP Report 384 (Harris 1997) provides an excellent treatise on use of plasma for steel bridges.

Plates are usually cut on water tables. The water helps capture smoke, reduce noise, capture glare, and generally improve quality. Some fabricators choose to submerge plate for plasma cutting, while others cut above the water. Neither method is detrimental to the steel.

It is rare that a mill edge will be used in fabrication. Mill edges are usually not square enough for welding, nor do they meet roughness criteria for free edges. Thus virtually all steel bridge plate edges are cut. Use of bar stock is an exception.

Good quality is readily achieved with both oxy-acetylene and plasma cutting. As-cut conditions are usually suitable for bridge service or welding, but workmanship problems can occur if equipment is not properly maintained.

2.4.2 Bolt Hole Preparation and Assembly

There are many ways to accomplish bolt holes. On the basis of their equipment and workforce skills, the fabricator will choose the means that will ensure proper connection fit and hole quality with the least amount of effort. To the extent possible, the fabricator will use CNC equipment to make bolt holes.

Traditionally, subsize holes were drilled or punched in individual plates, and then the holes were reamed to final size with the connection plies together. This is effective but requires a significant amount of component movement to join the plies. Preferably, the fabricator will use CNC equipment to drill or punch holes to final size in one step. Many connections can be effectively accomplished using this approach, and the fabricator can put a sample of plates together to demonstrate the proficiency of this process.

Depending on the complexity of the bridge, the relative stiffness of the bridge members, the capability of his/her equipment, and owner rules, the fabricator may elect to assemble components to accomplish holes. If so, the fabricator will support members in the no-load condition and set members to replicate field elevations, either vertically or horizontally. Assemblies typically include members from bearing to bearing, though on larger or heavily curved structures this is not always possible (Figure 2.7).

FIGURE 2.7 Girders in line assembly. Such assembly may be accomplished with the girders vertical, as shown, or horizontal, in the "lay down" position.

2.4.3 Welding

Fundamentally, welding revolutionized steel bridges. Early bridge engineers pushed steel spans further by stitching shapes and plates with rivets: large I-shapes were created by joining webs to flanges with angles. This technique was effective but did not facilitate curves for camber or sweep. It is the welding that allows ready construction of curved and cambered girders that comprise modern steel bridges.

Of the many types of welding available, arc welding is the primary source of welding used in steel bridges. There are four arc welding processes used in modern steel bridge fabrication:

Shielded metal arc welding: Also known as "stick" welding, accomplished using hand-held welding rods covered with cellulose, which burns off and creates shielding during welding. This is the earliest welding process used in bridge fabrication.

Submerged arc welding (SAW): Spools feed wire to the weld puddle, which is submerged in powder flux. This process is the most popular in use for steel bridge fabrication because it allows use of very heavy wires with high heat input and deposition rates that facilitate the accomplishment of long fillet welds (such as web-to-flange welds) and heavy weldments (such as flange butt splices).

Flux-cored arc welding (FCAW): Spools feed cored wire to the weld puddle, which is shield by just the flux that is in the core of the wire, or by a combination of this flux and gas.

Gas-metal arc welding (GMAW): Like FCAW, spools feed wire to the weld puddle, but in this case shielding is only by gas. Some GMAW wires are cored, but these cores contain alloy fillers and not flux.

The heat generated in an electric arc is a remarkable phenomenon of nature. Arc welding harnesses the power of the arc, using it to melt the tip of welding rod at one end of the arc and the base metal at the other. Liquid metal from the electrode then joins melted base metal in a molten pool that, using the proper technique, cools and solidifies quickly into a permanent bond of extraordinary strength, ductility, and toughness (Figure 2.8).

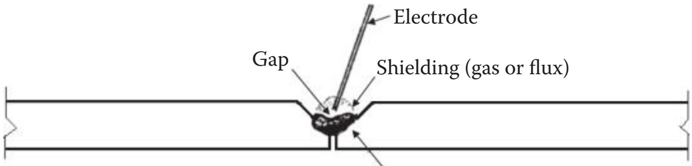

FIGURE 2.8 As shown in this idealized diagram, in arc welding, electric current moves through the electrode to the work and jumps the gap to produce the arc. The arc melts the tip of the electrode and the local base metal, which combine to form the weld puddle. Shielding protects the arc and weld pool from contaminants (notably air) to help ensure sound weld metal.

Achieving a good weld requires proper technique, including the following key attributes:

Shielding: The welding arc must be protected from atmosphere to preclude contamination, particularly by hydrogen. Small amounts of hydrogen reduce ductility; larger amounts of hydrogen result in porosity; still larger amounts result in cracks.

Alloy content: Like steel, the electrode can be designed with any variety of alloy constituents to produce the desired strength, ductility, toughness, and corrosion resistance.

Welding parameters: The shape and size of the molten pool is controlled by the welding amperage, voltage, travel speed, electrode angle and size, polarity, and the gap between the electrode and the work. Control of the molten pool impacts many factors that influence the performance of the weld, including the amount of penetration, the amount of base metal melted and contributed to the joint, the properties of the heat-affected zone, and the size and orientation of the weld metal grains that form as the pool cools.

Base metal cleanliness: Generally base metal should be clean of contaminants that can comprise weld integrity, particularly sources of hydrogen such as oil and grease. However, steel need not be blast cleaned, and light rust is also not deleterious. Mill scale can also be welded over, particularly using electrodes designed for this application.

Joint preparation: The weld joint has a significant effect on the size and shape of the molten weld pool. For groove welds, the fabricator will design the joint to optimize the equipment and processes while maintaining weld integrity.

For full-penetration welding, the American Weld Society (AWS) Bridge Welding Code (AASHTO/AWS 2011) provides many prequalified joint options. Under AWS rules for welding, use of these joints does not require further testing for use, though there may be additional tests required to qualify a welding procedure for other reasons. Fabricators may choose from among these prequalified joints or design and test their own joint.

Preheat: Preheat helps overcome local restraint to achieve welds of good integrity. Use of preheat may or may not be necessary to accomplish a good weld depending on the consumable being used and the joint conditions, especially material thickness: thicker materials provide a bigger heat sink for less tolerance of local restraint. The AWS welding codes have minimum preheat requirements for many prequalified applications, but fabricators may qualify procedures with less or no preheat.

Backgouging: Air carbon arc gouging is an effective and common means for quickly, if roughly, gouging the metal away. In this process, an arc between a carbon electrode melts the metal and a simultaneous air jet blows the melted material away. It is analogous to grinding, which grinds away metal more discretely at lower volumes.

Most full-penetration welds are accomplished from both sides of the plate. After a fabricator has welded the front side of a full-penetration joint, the plates being joined will be turned over, and the back side of the joint may be back-gouged (usually air carbon arc gouging process) before welding to help ensure that welding will be performed on sound metal at the weld root. This is backgouging, and it is required for prequalified full-penetration joints. The fabricator may feel that backgouging is not necessary, in which case they will need to qualify the procedure and the associated joint.

There are three basic weld types used in bridges: complete joint penetration (CJP) groove welds, partial joint penetration (PJP) groove welds, and fillet welds. Generally, fillet welds are more economical than groove welds and should be used when the design allows a choice. Partial penetration groove welds are often avoided in bridges because they do not perform well in fatigue, but they are a good choice when the design permits.

Groove welds are usually accomplished with stringer passes, that is, multiple welding passes that run along the length of the joint, filling the groove. SAW is the process of choice for groove welds because it offers large passes, which in turn minimize the total number of passes and help facilitate a clean weld.

Electroslag welding (ESW) offers significantly improved efficiency over SAW for thick full-penetration welds. Using ESW, one can accomplish groove welds in a single pass regardless of thickness; hence, the thicker the plate to be welded, the greater the improvement over a stringer pass welding. ESW is accomplished in the vertical-up position: a climbing column of weld metal fills a void bounded by the two plates to be joined on two sides and by copper shoes on the other two sides. The shoes retain the weld metal until it solidifies and, being water-cooled, help control welding temperature. In the 1970s, the original version of ESW was banned from bridge tension joints by the Federal Highway Administration (FHWA) due to concerns about heat-affected zone toughness; however, FHWA-sponsored research improved the process. The moratorium was lifted, and now ESW is making a comeback to steel bridge fabrication.

Fillet welds are usually accomplished in the horizontal position. In this position, gravity pulls larger passes out of shape, so the largest fillet welds that can consistently be accomplished in a single pass are 5/16 or 3/8 in., depending on the application, the process, the fabricator's equipment, and the fabricator's proficiency. Fabricators will mechanize or automate fillet welding to the extent possible.

2.4.4 Heat Curving and Heat Straightening

Heat is a useful tool for bending and shaping steel. Steel expands when it is heated, and it shrinks more than this expansion when it cools. Hence, fabricators apply heat to inside of the curve or bend that they want to introduce. To heat-curve a girder to introduce sweep, the fabricator will heat the inside of the curve. One common approach is to lay the girder on its side, with the inside curve of the girder facing up, and then apply strips of heat on the flange edges on this top side. In this way, the flanges shrink to the inside of the curve, and the weight of the girder contributes to the curving. Another approach would be to apply vee heats at discrete points on the flanges, with the open part of the vee facing the inside of the curve.

Heat is also useful for straightening, with heat applied in such a way to contract bent steel and bring it back to flat or straight. Straightening can be applied in the shop if needed, but it is more commonly used in the field to repair sections that have been damaged. It takes considerable experience to perform such repair on complex sections, but it is readily doable.

Typically, fabricators prefer to use as much heat as possible to accomplish bending or curving as quickly as possible. To avoid a phase shift molecular change in the steel, the applied heat should be kept below the steel phase-shift temperature. Usually a maximum limit of 1200°F is observed to provide a comfortable margin; the surface temperature of the steel must be continually monitored to ensure the steel is not overheated. Pressure may or may not be used in combination with heat; for example, as mentioned above, girder weight is often combined with heat to facilitate girder heat curving for sweep.

2.4.5 Coatings

Common modern steel bridge coatings come in four broad categories:

1. Weathering steel—no coating, except, possibly, for paint below joints
2. Zinc primer–based paint system
3. Galvanizing
4. Metalizing

In most environments, zinc coatings provide sufficient protection for the life of the structure (Figure 2.9). Zinc in the bridge coating provides cathodic protection to the steel, serving as a sacrificial anode.

Good cleaning is the key to successful coating application and long-term performance. Long-lasting protection with minimal maintenance depends on good adherence, which in turn demands clean steel with sufficient anchor pattern. Steel structure cleaning standards are established by the Society for Protective Coatings (SSPC), and two are common in steel bridges:

1. SSPC-SP6: Commercial Blast Cleaning—this is the standard usually required for bare weathering steel. Weathering steel does not need cleaning to ensure adherence of a subsequent coating, but it is good to clean the fascia surfaces to achieve a uniform appearance in service.
2. SSPC-SP10: Near-White Blast Cleaning—this is the standard usually required for zinc primers. Achieving this level of cleanliness will also ensure that sufficient anchor patter is achieved.

FIGURE 2.9 Roughly half of the steel bridges produced today are painted (the balance are unpainted and comprise weathering steel).

Fabricators generally have blast cabinets to accomplish blast cleaning, but this can also be done by hand. Note that SP10 is not the highest level of cleaning; rather, the highest level is SSPC SP5, which is known as “white blast.” Reaffirming, near-white, or SSPC-SP10, is sufficient for zinc primer; white blast cleaning is not necessary. Note, however, that once a girder is blasted, it will not stay clean indefinitely, and so the steel cannot be cleaned too long before the primer is applied. Though some owners specify an anchor pattern for use with zinc primers, achieving an SP10 will ensure that a sufficient anchor pattern is achieved.

2.4.5.1 Zinc Primer–Based Systems

In zinc primer–based systems, the zinc primer provides cathodic protection to the steel, and then a top coat protects the primer and provides an aesthetic look. Zinc primer systems come in two classes: organic and inorganic. Top coats bind readily to organic primers, resulting in a two-coat system. Inorganic systems require an intermediate coat to bind the top coat to the primer, so inorganic systems typically have three coats.

Both organic and inorganic zinc primer systems perform excellently, and studies show that they perform comparably to each other. Some feel that inorganic primer is superior because it is a very hard coating; others prefer organic coatings, particularly for field painting, because they are more forgiving in application. Both offer about the same content of zinc and therefore the same protection.

Proper curing is essential for zinc primer system performance, and curing has a notable impact on fabrication throughput. Primers require moisture to cure; the amount varies from coating to coating, but inorganic zinc primers generally require more time to cure, requiring a longer wait between application of the primer and the subsequent coatings. Of note, the epoxy intermediate coat, which binds the urethane to the inorganic primer, acts as a moisture barrier; hence once the epoxy intermediate coat is applied, there will be no more curing of the primer.

Zinc primers are suitable for bolted connections. Most will achieve a class B friction condition, though others are only suitable for class A. The friction condition is established by testing prescribed in the bolting specification found in the RCSC (2009). Generally testing is conducted by a private laboratory on behalf of the paint manufacturer, and the paint manufacturer certifies the results.

Customary practice is to use the primer on the faying surface and put top coats on the outside surfaces of splice plates. Top coats can be applied in the shop or field, and both practices are common. Tops of top flanges are usually left unpainted because they will be under concrete. However, it is a good idea to use a light brush coating of primer on the top flange for protection during storage and to preclude rust forming and running onto the painted surfaces, which is unsightly and difficult to remove well.

2.4.6 Nondestructive Examination

Rules and practices for nondestructive examination (NDE) in bridges are codified in the *Bridge Welding Code* (AASHTO/AWS D1.5, 2011). This code prescribes acceptance criteria and technique requirements, including reference to other standards. Chapter 12 presents detailed discussion on nondestructive testing (NDT) and NDE. NDE technicians are qualified to American Society of Nondestructive Testing (ASNT) requirements found in ASNT-TC-1A, which defines three qualification levels.

2.4.6.1 Level III

All shop NDE practices must fall under the auspices of an ASNT Level III. The Level III qualification is achieved through rigorous testing administered by ASNT. The Level III oversees NDE methods and personnel qualifications in the shop. The Level III can provide these services on a contract basis; the technician does not need to be an employee of the shop.

2.4.6.2 Level II

NDE is performed by Level II technicians. To achieve Level II certification, the technician must have experience with the method and must pass practical and written examinations in the method in question. Testing is administered by the shop Level III; the Level III must affirm that the NDE technician has passed the required test and has the minimum experience required.

2.4.6.3 Level I

This is the trainee level of certification. Trainees can help testing or perform part of the testing under the close supervision of a Level II, but they do not have the authority to conduct the testing on their own or take responsibility for the results.

There are four NDE methods commonly used in steel bridge shop inspection. Two of them are volumetric (i.e., examine through the thickness of the weld), which are radiography (RT) and ultrasonic testing (UT); and the other two are surface inspections, which are magnetic particle testing (MT) and dye penetrant testing (PT). D1.5 provides inspection frequencies and acceptance criteria for RT and UT of main member butt splices and MT of main member fillet welds. All four methods are also used to supplement fabrication at the fabricator's discretion and for repair acceptance at the owner's discretion.

Radiography (RT): Radiography is x-ray testing of bridge welds. There is prescribed RT testing for web and flange butt splices in D1.5; RT is also sometimes used in base metal repairs. It is well suited to testing flat plates, including thickness transitions, but is not well suited to testing angular geometries where a uniform amount of x-ray energy cannot be applied through all of the metal.

In RT, a film is placed under the joint being tested, and the x-ray source is placed a few feet above the joint. X-rays are generated by an x-ray tube or an isotope; the x-ray shoot through the joint for the right amount of time to expose the x-ray film properly. A standard hole or wire-type image quality indicator is placed on the film during exposure; later, when the film is examined, the appearance of the wire or hole on the film verifies that the film was properly exposed. Owing to the complications of the method and the laws associated with maintaining and handling x-ray sources, fabricators often contract out this testing.

The RT acceptance criteria are based on workmanship and not on fit-for-purpose. The maximum allowable flaw size is different for splices on tension and compression components, so designs must indicate the stress condition of splices to facilitate this inspection. The allowable flaw size is small, varying from 1/16 in. on 1/2 in. plate to 7/16 in. on 4 in. plate for joints in tension, and about double for compression joints. The allowance also varies with proximity to the edge of the plate and proximity to other flaws.

Traditional x-ray is performed on film, but the steel industry is moving (belatedly) to digital x-ray. Two types are common: computed radiography (CR), by which a plate is used to capture the x-ray image, and then the plate is processed, and digitized radiography (DR), by which the x-rays are transformed into light after passing through steel, and then the light image is immediately captured digitally, with processing of a plate not required. Transition to these methods will be straightforward because the acceptance criteria for digital radiography are the same as it is for film radiography.

Ultrasonic testing (UT): Ultrasonic testing is examination of full-penetration welds or just plates with sound. Under this method a prescribed amount of sound is sent through the area of interest by a probe; the sound reflects back and forth through the plate repeatedly, and the probe picks up the return signals, which are displayed on an oscilloscope. If there are no defects in the metal, then the return signals are uniform; a discontinuity in the metal will break the uniformity of the sound reflections and show up on the oscilloscope.

In UT acceptance is based on sound loss. When a flaw is discovered, it is assigned a class, A, B, C, or D, and acceptance is based on the class and flaw location. Probes that direct sound at different angles are used to ensure flaws are examined from the right critical direction, including 45°, 60°, and 70°, as well as straight beam (or zero degrees). UT technicians can locate flaws by simple trigonometric calculation using sound path length to the defect and the probe angle.

UT acceptance is based on calibration of the UT equipment to a known defect size in a defined calibration block. In this way it is similar to RT, but the allowable classes do not correlate directly to allowable RT flaws. Like RT, the allowable defects are based on workmanship and not on fitness-for-purpose.

UT is much simpler to use than RT, so fabricators often supplement RT with UT. When a defect is found by RT, the fabricator often follows up RT with UT to help pinpoint the location of the flaw within the depth of the steel (RT is depth neutral), and during repair UT may be used to help ensure that defects are cleaned properly. UT is also effective for verifying the integrity of plates if a question arises about such integrity.

Magnetic particle testing (MT): Under this method, two magnets of prescribed strength are placed on the steel about the area of interest, and colored iron particles are sprinkled on the area of interest, which is usually a fillet weld. The particles are then gently blown away. If discontinuities are present and open to the surface, these discontinuities disrupt the surface magnetic field and cause particles to gather at the discontinuity, thus revealing a crack.

A sampling of about 10% of main member fillet and PJP welds is required by D1.5. Fabricators also use MT to help accomplish repairs. For example, when a defect in a plate has been removed and preparation for rewelding has been completed, the fabricator may check the remediated surface with MT to ensure that there are no surface discontinuities before beginning welding.

Dye penetrant testing (PT): There is no prescribed PT in the *Bridge Welding Code*, but this method is very useful in the shop for close-up checking of surface conditions. For example, a fabricator might use PT technique to check a weld joint after backgouging to ensure there is nothing in the prepared surface to compromise the integrity of the weld. Like MT, PT is a good technique for finding surface discontinuities that are not visible to the naked eye. Under this method, the colored dye is sprayed on the surface in question, left in place for a bit of time, and then wiped off. Then, a white developer is sprayed on the surface; if a discontinuity is present, the dye will have penetrated the discontinuity and will then appear in the developer.

2.5 Shipping

Shipping is an important consideration for cost-effective steel bridge design: it must be possible to deliver bridge members to the bridge site.

There are three general shipping options: truck (Figure 2.10), rail, and barge, and the flexibility and therefore cost of the methods follows that order. Truck shipping allows ready access into all steel bridge shops; fabricators frequently use trucks to move work around the shop and yard during fabrication. Once a truck is loaded with finished members, it can go right to the jobsite without further handling of the work, offering the best schedule and keeping costs down. Truck shipping is limited by highway clearances, particularly underpass heights along the shipping route, and to lesser extent by weight.

Barges are capable of carrying extraordinary loads—hundreds of tons and tens of feet high. Use of barge requires (1) access to the bridge by barge, and (2) access to the barge from the shop. If the first condition is satisfied, then barge shipment is a possibility, but building a bridge that requires barge delivery significantly increases cost.

For larger members, the fabricator will analyze loads and design the proper support conditions to ensure safety, stability, and load distribution.

FIGURE 2.10 Trucking is the most common approach used for shipping steel girders.

2.6 Concluding Remarks

Steel bridge fabrication can be divided into two phases: (1) prefabrication, featuring shop drawing development and material procurement, and (2) actual fabrication, when steel is cut, fitted, welded, drilled, coated, and shipped. Achievement of a project that is successful in quality, schedule, and cost depends on good support for the detailer from the designer and owner during prefabrication, and use of standard details and allowance of standard practices during actual fabrication. This treatise provides an overview of the values important to fabricators, but for a truly successful project, this introductory knowledge should be combined with good communication among the fabricator, detailer, designer, erector, and owner.

References

AASHTO/AWS. 2011. *Bridge Welding Code, D1.5M/D1.5:2010*, 6th Edition, with 2011 AASHTO Interim Revisions, American Welding Society, Miami, FL.

AASHTO/NSBA. 2000. *Shop Detail Drawing Review and Approval Guideline, G1.1*, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO/NSBA. 2002. *Shop Detail Drawing Presentation Guidelines, G1.3*, AASHTO/NSBA Steel Bridge Collaboration, AASHTO Publication Code: NSBASDDP-1, American Association of State Highway and Transportation Officials, Washington, DC.

Harris, I. D. 1997. *Plasma Arc Cutting of Bridge Steels*, NCHRP Report 385, Transportation Research Board, Washington, DC.

RCSC. 2009. *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Research Council on Structural Connections, Chicago, IL.

3

Concrete Bridge Construction

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3.1 Introduction

This chapter focuses on the principle and practices related to construction of concrete bridges in which construction engineering contributes greatly to the successful completion of the projects. We first present the fundamentals of construction engineering and analyze the challenges and obstacles involved in such processes and then introduce the problems in relation to design, construction practices, project planning, scheduling, and control, which are the basis of future factorial improvements in effective construction engineering in the United States. Finally, we discuss prestressed concrete, high-performance concrete (HPC), and falsework in some details.

3.2 Effective Construction Engineering

The construction industry is a very competitive business, and many companies that engage in this marketplace develop proprietary technology in their field. In reality, most practical day-to-day issues are very common to the whole industry. Construction engineering is a combination of art and science and has a tendency to become more the art of applying science (engineering principles) and approaches to the construction operations (Blank and Blank 1995; Blank 1996). Construction engineering includes design, construction operation, and project management. The final product of design team effort is to produce drawings, specifications, and special provisions for various types of bridges. A fundamental part of construction engineering is construction project management (project design, planning, scheduling, controlling, etc.).

Planning starts with analysis of the type and scope of the work to be accomplished and selection of techniques, equipment, and labor force. Scheduling includes the sequence and interrelation of operations both at a job site and with external aspects, as well as allocation of man power and equipment. Controlling consists of supervision, engineering inspection, detailed procedural instructions, record maintenance, and cost control. Good construction engineering analysis produces more valuable, effective, and applicable instructions, charts, schedules, and so on.

The objective is to plan, schedule, and control the construction process such that every construction worker and every activity contributes to accomplishing tasks with minimum waste of time and money and without interference. All construction engineering documents (charts, instructions, and drawings) must be clear, concise, definitive, and understandable by those who actually perform the work. As mentioned earlier in this section, the bridge is the final product of design team efforts. When all phases of construction engineering are completed, this product—the bridge—is ready for service loading. In all aspects of construction engineering, especially in prestressed concrete, design must be integrated for the most effective results. The historical artificial separation of the disciplines—design and construction engineering—was set forth to take advantage of the concentration of different skills in the workplace. In today's world, the design team and construction team must be members of one team, partners with one common goal. That is the reason partnering represents a new and powerful team-building process, designed to ensure that projects become positive, ethical, and win-win experiences for all parties involved.

The highly technical nature of a prestressing operation makes it essential to perform preconstruction planning in considerable detail. Most problems associated with prestressed concrete could have been prevented by properly planning before the actual construction begins. Preconstruction planning at the beginning of the projects ensures that the structure is constructed in accordance with the plans, specifications, and special provisions and also helps detect problems that might arise during construction. It includes (1) discussions and conferences with the contractor; (2) review of the responsibilities of other parties; and (3) familiarization with the plans, specifications, and special provisions that relate to the planned work, especially if there are any unusual conditions. The preconstruction conference might include such items as scheduling, value of engineering, grade control, safety and environmental issues, access and operation considerations, falsework requirements, sequence of concrete placement, and concrete quality control (QC) and strength requirements. Preconstruction planning has been very profitable and in many cases has resulted in substantial reduction of labor costs. More often, in prestressed concrete construction the details of tendon layout, selection of prestressing system, mild-steel details, and so on, are left up to general contractors or their specialized subcontractors, with the designer showing only the final prestress and its profile and setting forth criteria. The contractors must understand the design consideration fully to select the most efficient and economical system. Such knowledge may, in many cases, provide a competitive edge, and construction engineering can play a very important role in it.

3.3 Construction Project Management

3.3.1 General Principles

Construction project management (Fisk 1997) is a fundamental part of construction engineering. It is a feat that few, if any, individuals can accomplish alone. It may involve a highly specialized technical field or science, but it always includes human interactions, attitudes and aspects of leadership, common sense, and resourcefulness. Although no one element in construction project management can create success, failure in one of the foregoing elements will certainly be enough to promote failure and escalate costs. Today's construction environment requires serious consultation and management of the following life-cycle elements: design (including specifications, contract clauses, and drawings), estimating, budgeting, scheduling, procurement, bid ability–constructability–operability (BCO) review, permits and licenses, site survey, assessment and layout, preconstruction and mutual understanding conference, safety, regulatory requirements, QC, construction acceptance, coordination of technical and special support, construction changes and modifications, maintenance of progress drawing (redlines), creating as-built drawings, and project records, among other elements.

Many construction corporations are becoming more involved in environmental restoration either under the Resource Conservation and Recovery Act or under the Comprehensive Environmental Response, Compensation, and Liability Act (otherwise commonly known as Superfund). This new involvement requires additional methodology and considerations by managers. Some elements that would otherwise be briefly covered or completely ignored under normal considerations may be addressed and required in a site's Specific Health and Environmental Response Plan (SHERP). Some elements of the SHERP may include site health and safety staff, site hazard analysis, chemical and analytical protocol, personal protective equipment requirements and activities, instrumentation for hazard detection, medical surveillance of personnel, evacuation plans, special layout of zones (exclusion, reduction, and support), and emergency procedures.

Federal government contracting places additional demands on construction project management in terms of added requirements in the area of submittals and transmittals, contracted labor and labor standards, small disadvantaged subcontracting plans, and many other contractual certification issues, among others. Many of these government demands are recurring elements throughout the life cycle of the project, which may require adequate resource allocation (man power) not necessary under the previous scenarios.

The intricacies of construction project management require the leadership and management skills of a unique individual who is not necessarily a specialist in any one of the aforementioned elements but who has the capacity to converse and interface with specialists in various fields (i.e., chemists, geologists, surveyors, mechanics, etc.). An individual with a combination of an engineering undergraduate degree and a graduate business management degree is most likely to succeed in this environment. Field management experience can substitute for an advanced management degree.

It is the purpose of this section to discuss and elaborate elements of construction project management and to relate some field experience and considerations. The information presented here will only stimulate further discussion and is not intended to be all-inclusive.

3.3.2 Contract Administration

Contract administration focuses on the relationships between the involved parties during the contract performance or project duration. Owing to the nature of business, contract administration embraces numerous postaward and preaward functions. The basic goals of contract administration are to ensure that the owner is satisfied and all involved parties are compensated on time for their efforts. The degree and intensity of contract administration will vary from contact to contact, depending on the size and complexity of the effort to be performed. Since money is of the essence, too many resources can add costs and expenditures to the project, whereas insufficient resources may also cost in loss of time, in inefficiencies, and in delays. A successful construction project management program is one that has the vision and flexibility to allocate contract administrative personnel and resources wisely and that maintains a delicate balance in resources necessary to sustain required efficiencies throughout the project life cycle.

3.3.3 Project Design

Project design is the cornerstone of construction project management. In this phase, concepts are drawn, formulated, and created to satisfy a need or request. The design is normally supported by sound engineering calculations, estimates, and assumptions. Extensive reviews are performed to minimize unforeseen circumstances, avoiding construction changes or modifications to the maximum extent possible in addition to verifying facts, refining or clarifying concepts, and dismissing assumptions. This phase may be the ideal time for identification and selection of the management team.

Normally, 33%, 65%, 95%, and 100% design reviews are standard practice. The final design review follows the 95% design review, which is intended to ensure that review comments have been either incorporated into the design or dismissed for consideration. Reviews include design analysis reviews and BCO reviews. It can be clearly understood from the nomenclature that a BCO encompasses all facets of a project. Bid ability relates to how the contact requirements are worded to ensure clarity of purpose or intent and understanding by potential construction contractors. Constructability concentrates on how components of the work or features of the work are assembled and how they relate to the intended final product. The main purpose of the constructability review is to answer questions such as whether a structure can be built in the manner represented in the contract drawings and specifications. Interaction between mechanical, civil, electrical, and other related fields is also considered here. Operability includes aspects of maintenance and operation, warranties, services, man power, and resource allocation during the life of the finished work.

The finished product of the design phase should include construction drawings illustrating dimensions, locations, and details of components; contract clauses and special clauses outlining specific needs of the construction contractor; specifications for mechanical, civil, and electrical or special equipment; a bidding and payment schedule with details on how parties will be compensated for work performed or equipment produced and delivered; responsibilities; and operation and maintenance (O&M) requirements. In many instances, the designer is involved throughout the construction phase for design clarification or interpretation, incorporation of construction changes or modifications to the project, and possible O&M reviews and actions. It is not uncommon to have the designer perform contract management services for the owner.

There are a number of computer software packages readily available to assist members of the management team in writing, recording, transmitting, tracking, safekeeping, and incorporating BCO comments. Accuracy of records and safekeeping of documentation regarding this process has proved to be valuable when a dispute, claim, design deficiency, or liability issue is encountered later during the project life cycle.

3.3.4 Planning and Scheduling

Planning and scheduling are ongoing tasks throughout the project until completion and occupancy occur by a certain date. Once the design is completed and the contractor selected to perform the work, the next logical step may be to schedule and conduct a preconstruction conference. Personnel representing the owner, designer, construction contractor, regulatory agencies, and any management/oversight agency should attend this conference. Among several key topics to discuss and understand, construction planning and scheduling is most likely to be the main subject of discussion. It is during this conference that the construction contractor may present how the work will be executed. The document here is considered the "baseline schedule." Thereafter, the baseline schedule becomes a living document by which progress is recorded and measured. Consequently, the baseline schedule can be updated and reviewed in a timely manner and becomes the construction progress schedule. The construction progress schedule is the means by which the construction contractor records progress of work, anticipates or forecasts requirements so that proper procurement and allocation of resources can be achieved, and reports the construction status of work upwardly to the owner or other interested parties. In addition, the construction contractor may use progress schedule information to assist in increasing efficiencies or to formulate the basis of payment for services provided or rendered and to anticipate cash flow requirements. The construction progress schedule can be updated as needed, or as mutually agreed to by the parties, but for prolonged projects it is normally produced monthly.

A dedicated scheduler, proper staffing, and adequate computer and software packages are important to accomplish this task properly. On complex projects, planning and scheduling is a full-time requirement.

3.3.5 Safety and Environmental Considerations

Construction of any bridge is a hazardous activity by nature. No person may be required to work in surroundings or under conditions that are unsafe or dangerous to his or her health. The construction project management team must initiate and maintain a safety and health program and perform a job hazard analysis with the purpose of eliminating or reducing risks of accidents, incidents, and injuries during the performance of the work. All features of work must be evaluated and assessed to identify potential hazards and implement necessary precautions or engineer controls to prevent accidents, incidents, and injuries.

Frequent safety inspections and continued assessment are instrumental in maintaining the safety aspects and preventive measures and considerations related to the proposed features of work. In the safety area, it is important for the manager to be able to distinguish between accidents/incidents and injuries. Lack of recorded work-related injuries is not necessarily a measure of how safe the work environment is on the project site. The goal of every manager is to complete the job in an accident/incident- and injury-free manner, as every occurrence costs time and money.

Today's construction operational speed, government involvement, and community awareness are placing more emphasis, responsibilities, and demands on the designer and construction contractor to protect the environment and human health. Environmental impact statements, storm water management, soil erosion control plans, dust control plans, odor control measures, analytical and disposal requirements, Department of Transportation (DOT) requirements for overland shipment, activity hazard analysis, and recycling are some of the many aspects that the construction project management team can no longer ignore or set aside. As with project scheduling and planning, environmental and safety aspects of construction may require significant attention from a member of the construction management team. When not properly coordinated and executed, environmental considerations and safety requirements can delay the execution of the project and cost significant amounts of money.

3.3.6 Implementation and Operations

Construction implementation and operations is the process by which the construction project manager balances all construction and contact activities and requirements to accomplish the tasks. The bulk of construction implementation and operations occur during the construction phase of the project. The construction project management team must operate in synchronization and maintain good communication channels to succeed in this intense and demanding phase. Many individuals in this field may contend that the implementation and operation phase of the construction starts with the site mobilization. Although it may be an indicator of actual physical activity taking place on site, construction implementation and operations may include actions and activities prior to the mobilization to the project site.

Here, a delicate balance is attempted to be maintained between all activities taking place and those activities being projected. Current activities are performed and accomplished by field personnel with close monitoring by the construction management staff. Near (~1 week ahead), intermediate (~2 to 4 weeks), and distant future (over 4 weeks) requirements are identified, planned, and scheduled to procure equipment and supplies, schedule work crews, and maintain efficiencies and progress. Coordinating progress and other meetings and conferences may take place during the implementation and operation phase.

3.3.7 Value Engineering

Some contracts include an opportunity for contractors to submit a value engineering (VE) recommendation. This recommendation is provided to either the owner or the designer. The purpose of VE is to promote or increase the value of the finished product while reducing the dollars spent or invested; in other words, to provide the desired function for the minimum costs. VE is not intended to reduce performance, reliability, maintainability, or life expectancy below the level required to perform the basic function. Important VE evaluation criteria are in terms of "collateral savings"—the measurable net reductions in the owner/agency's overall costs of construction, operations, maintenance, and/or logistics support. In most cases, collateral savings are shared between the owner/agency and the proponent of the VE by reducing the contract price or estimated cost in the amount of the instant contract savings and by providing the proponents of the VE a share of the savings by adding the amount calculated to the contract price or fee.

3.3.8 Quality Management

During the construction of a bridge, construction quality management (CQM) plays a major role in QC and quality assurance (QA). CQM refers to all control measures and assurance activities instituted by the parties to achieve the quality established by the contract requirements and specifications. It encompasses all phases of the work, such as approval of submittals, procurements, storage of materials and equipment, coordination of subcontractor activities, and the inspections and the tests required to ensure that the specified materials are used and that installations are acceptable to produce the required product. The key elements of the CQM are the contractor quality control (CQC) and QA. To be effective, there must be a planned program of actions and lines of authority and responsibilities must be established. CQC is primarily the construction contractor's responsibility, whereas QA is primarily performed by an independent agency (or other than the construction contractor) on behalf of the designer or owner. In some instances, QA may be performed by the designer. In this manner, a system of checks and balances is achieved, minimizing the conflicts between quality and efficiency that are normally developed during construction. Consequently, CQM is a combined responsibility.

In the CQC, the construction contractor is primarily responsible for (1) producing the quality product on time and in compliance with the terms of the contract, (2) verifying and checking the adequacy of the construction contractor's QC program of the scope and character necessary to achieve the quality of construction outlined in the contract, and (3) producing and maintaining acceptable record of its QC activities. In the QA, the designated agency is primarily responsible for (1) establishing standards and QC requirements; (2) verifying and checking adequacy of the construction contractor's QC (QA for acceptance), performing special tests and inspections as required in the contract, and determining that reported deficiencies have been corrected; and (3) assuring timely completion.

3.3.9 Partnership and Teamwork

A great deal of construction contract success attributes to partnering. Partnering should be undertaken and initiated at the earliest opportunity during the construction project management cycle. Some contracts may have a special clause, which is intended to encourage the construction contractor to establish clear channels of communication and effective working relationships. The best approach to partnering is for the parties to volunteer to participate.

Partnering differs from the team-building concept. Team building may encourage establishing open communications and relationships when all parties share liabilities, risk, and money exposure, but not necessarily share costs of risks. The immediate goal of partnering is to establish mutual agreements at the initial phase of the project on the following areas: identification of common goals, identification of common interests, establishment of communication, and establishment of lines of authority and decision-making commitment to cooperative problem solving, among others.

Partnering takes the elements of luck, hope, and personality out of determining project success. It facilitates workshops in which stakeholders in a specific project or program come together as a team, which results in breakthrough success for all parties involved. To increase awareness in partnership and teamwork in the Department of the Army, Philadelphia district, Corps of Engineers, Dr. Michael Blank and Capt. Jeff Barson, have initiated the "Partnering Team Award." This award was approved by the commander of the army, Corps of Engineers (Philadelphia district).

It aimed to recognize teams (internal and external to the district) who have achieved the best exemplification of partnering and team approach to the mission accomplishment. Now it has been given annually to the team of teams, which embodies the spirit, code, and principles of effective partnering while fostering a customer-focused relationship that produces the best-quality work ahead of schedule, under budget, and with the newest and most innovative technology available.

Over the past few years, the partnering process has been well defined in published papers by different authors, equally coming from both the government and the private sectors (Kubai 1994; Godfrey 1995; Schriener 1995; Tri-Service Committee 1996; Blank et al. 1997).

Air Force, Navy and the Army have had a positive experience and have acquired skilled expertise in the implementation of partnering in their various civil works (bridges, road construction, etc.), military, and support for other (U.S. Environmental Protection Agency, Federal Aviation Administration, etc.) projects.

Another success example is the Office of Structure Construction (OSC) of the California Department of Transportation (Caltrans). Richmond field office (part of OSC) with his Structures Engineer Simon A. Blank and partnering team of Kiewit Pacific Company (a Kiewit company) contributed to achieve effective partnering in a work, which resulted not only of high quality, but was delivered in the safest manner possible in widening bridges at Highway I-80 corridor in cities of Berkeley and Emeryville in San Francisco Bay Area.

In partnership with the districts and other clients, the OSC does the following to achieve the best results in partnering:

- Administers and inspects the construction of the Caltrans transportation structures and related facilities in a safe and efficient manner
- Provides specialized equipment and training, standards, guidelines, and procedural manuals to ensure consistency of inspection and administration by statewide OSC staff
- Provides consultations on safety for OSC staff and district performing structure construction inspection work
- Conducts reviews and provides technical consultation and assistance for trenching and shorting temporary support and falsework construction reviews
- Provides technical recommendations on the preparation of structure claims and the contract change orders (CCOs)
- Provides construction engineering oversight on structure work on non-state-administrated projects
- Conducts BCO review

3.3.10 Project Completion and Turnover of Facility

Success in construction project management may be greatly impacted during project completion and turnover of the facilities to the user or owner. The beginning of the project completion and turnover phase may be identified by one of the following: punch list developed, prefinal inspections scheduled, support areas demobilized, and site restoration initiated, just to mention a few. Many of the problems encountered during completion of this last phase may be avoided or prevented with proper user or owner participation and involvement during the previous phases, particularly during the construction phase where changes and modifications may have altered the original design. A good practice in preventing conflicts during the completion and turnover of the facilities is to invite the owner or user to all construction progress meetings and acceptance inspections. In this manner, the user or owner is completely integrated during the construction with ample opportunity to provide feedback and be part of the decision-making process. In addition, by active participation, the owner or user is being informed and made aware of changes, modifications, and/or problems associated with the project.

3.4 Major Construction Considerations

Concrete bridge construction involves site investigation; structure design; selection of materials—steel, concrete, aggregates, and mix design; workmanship of placement and curing of concrete; and handling and maintenance of the structure throughout its life. Actually, site investigations are made of any structure, regardless of how insignificant it may be. The site investigation is very important for intelligent design of the bridge structures and has significant influence on selection of the material and mix. A milestone is to investigate the fitness of the location to satisfy the requirements of the bridge structure. Thus, investigation of the competence of the foundation to carry the service load safely and an investigation of the existence of forces or substances that may attack the concrete structure can proceed. Of course, the distress or failure may have several contributing causal factors: unsuitable materials, construction methods, and loading conditions; faulty mix design; design mistakes; conditions of exposure; curing condition; or environmental factors.

3.5 Structural Materials

3.5.1 Normal Concrete

3.5.1.1 Important Properties

Concrete is the only material that can be made on site and is the most dependable and versatile construction material practically used in bridge construction (Naway 2008). Good durable concrete is quality concrete that meets all structural and aesthetic requirements for a period of structure life at minimum cost. We are looking for properties such as workability in the fresh condition; strength in accordance with design, specifications, and special provisions; durability; volume stability; freedom from blemishes (scaling, rock pockets, etc.); impermeability; economy; and aesthetic appearance. Concrete, when properly designed and fabricated, can actually be crack-free, not only under normal service loads, but also under moderate overload, which is very attractive for bridges that are exposed to an especially corrosive atmosphere.

The codes and specifications usually specify the minimum required strength for various parts of a bridge structure. The required concrete strength is determined by design engineers. For cast-in-place concrete bridges, a compressive strength of 3250–5000 psi (22–33 MPa) is usual. For precast structures, compressive strength of 4000–6000 psi (27–40 MPa) is often used. For special precast, prestressed structures compressive strength of 6000–8000 psi (40–56 MPa) is used by Caltrans: whenever the 28-day compressive strength shown on the plans is greater than 3600 psi, the concrete shall be designated by strength.

If the plans show a 28-day compressive strength that is 4000 psi or greater, an additional 14 days will be allowed to obtain the specified strength. The 28-day compressive strength shown on the plans that is 3600 psi or less is shown for design information only and is not a requirement for acceptance of the concrete. Other properties of concrete are related to the strength, although not necessarily dependent on the strength.

Workability is the most important property of fresh concrete and depends on the properties and proportioning of the materials: fine and coarse aggregates, cement, water, and admixtures. Consistency, cohesiveness, and plasticity are elements of workability. Consistency is related to the fluidity of mix. Just adding water to a batch of concrete will make the concrete more fluid or "wetter," but the quality of the concrete will diminish. Consistency increases when water is added and an average of 3% in total water per batch will change the slump about 1 in. (2.54 cm). Both research and practice show that workability is a maximum in concrete of medium consistency, between 3 in. (7.62 cm) and 6 in. (15.24 cm) slump. Very dry or wet mixes produce less workable concrete. Use of relatively harsh and dry mixes is allowed in structures with large cross sections, but congested areas containing much reinforcement steel and embedded items require mixes with a high degree of workability.

A good and plastic mixture is neither harsh nor sticky and will not segregate easily. Cohesiveness is not a function of slump, as very wet (high-slump) concrete lacks plasticity. On the other hand, a low-slump mix can have a high degree of plasticity. A harsh concrete lacks plasticity and cohesiveness and segregates easily.

Workability has a great effect on the cost of placing concrete. Unworkable concrete not only requires more labor and effort in placing, but also produces rock pockets and sand streaks, especially in small congested forms. It is a misconception that compaction or consolidation of concrete in the form can be done with minimum effort if concrete is fluid or liquid to flow into place. It is obvious that such concrete will flow into place but segregate badly, so that large aggregate will settle out of the mortar and excess water will rise to the top surface. Unfortunately, this error in workmanship will become apparent after days, even months later, showing up as cracks, low strength, and general inferiority of concrete. The use of high-range water-reducing admixtures (superplasticizers) allows placing of high-slump, self-leveling concrete. They increase strength of concrete and provide great workability without adding an excessive amount of water. An example of such products used in Caltrans is PolyHeed 997, which meets the requirements for a type A, water-reducing admixture specified in American Society for Testing and Materials (ASTM) C 494-92, Corps of Engineers CRD-C 87-93, and American Association of State Highway and Transportation Officials 194-87, the standard specifications for chemical admixtures for concrete.

3.5.1.2 Special Consideration for Cold Weather Construction

Cold weather can damage a concrete structure by freezing of fresh concrete before the cement has achieved final set and by repeated cycles of freezing of consequent expansion of water in pores and openings in hardened concrete. Causes of poor frost resistance include poor design of construction joints and segregation of concrete during placement; leaky formwork; poor workmanship, resulting in honeycomb and sand streaks; and insufficient or absent drainage, permitting water to accumulate against concrete. To provide resistance against frost, it is suggested to design adequate drainage. If horizontal construction joints are necessary, they should be located below the low-water or above the high-water line about 2–3 ft. (0.6–1 m). Previously placed concrete must be cleaned up. The concrete mix should have 7% (maximum) air for ½ in. (12.7 mm) or ¾ in. (19 mm) (maximum) aggregate, ranging down to 3%–4% for cobble mixes. It is essential to use structurally sound aggregates with low porosity. The objective of frost-resistant concrete mix is to produce good concrete with smooth, dense, and impermeable surface. This can be implemented by good construction technique used in careful placement of concrete as near as possible to its final place, avoiding segregation, sand streaks, and honeycomb under proper supervision, QC, and QA.

Sudden changes in temperature can stress concrete and cause cracking or crazing. A similar condition exists when cold water is applied to freshly stripped warm concrete, particularly during hot weather.

For the best results, the temperature difference should not exceed 25°F between concrete and curing water. In case when anchor bolt holes were left exposed to weather, filled with water, freezing of water exerted sufficient force to crack concrete. This may happen on the bridge pier cap under construction.

3.5.1.3 Concrete Reinforcement and Placement

The optimum condition for structural use is a medium slump of concrete and compaction by vibrators. A good concrete with low slump for the placing conditions can be ruined by insufficient or improper consolidation. Even workable concrete may not satisfy the needs of the bridge structure if it is not properly consolidated, preferably by vibration. An abrupt change in size and congestion of reinforcement not only makes proper placing of concrete difficult but also causes cracks to develop. Misplacement of reinforcement within concrete will greatly contribute to development of structure cracks. The distress and failure of concrete are mostly caused by ignorance, carelessness, wrong assumptions, and so on.

3.5.1.4 Concrete Mix and Trail Batches

The objective of concrete mix designs and trail batches is to produce cost-effective concrete with sufficient workability, strength, and impermeability to meet the conditions of placing, finishing characteristics, exposure, loading, and other requirements of bridge structures. A complete discussion of concrete mixes and materials can be found in many works such as the *Concrete Construction Engineering Handbook* by Naway (2008). The purpose of trail batches is to determine strength, water–cement ratio, combined grading of aggregates, slump, type and proportioning of cement, aggregates, entrained air, and admixtures as well as scheduling of trail batches and uniformity. Trail batches should always be made for bridge structures, especially for large and important ones. They should also be made in cases where there is no adequate information available for existing materials used in concrete mixes, and they are subjected to revisions in the field as conditions require.

3.5.1.5 Consideration to Exposure Condition

Protection of waterfront structures should be considered when they are being designed. Designers often carefully consider structure aesthetic aspects without consideration of exposure conditions. Chemical attack is aggravated in the presence of water, especially in transporting the chemicals into the concrete through cracks, honeycombs, or pores in surfaces. Use of chamfers and fillers is a good construction practice. Chamfering helps prevent spalling and chipping from moving objects. Fillets in reentrant corners eliminate possible scours or cracking. Reinforcement should be well covered with sound concrete and in most cases 3 in. (7.62 cm) coverage is specified. First-class nonreactive and well-graded aggregates in accordance with the Uniform Building Code standard should be used. Cement type II or type Y with a low amount of C3 should be used. Careful consideration should be given to the use of an approved pozzolan with a record of successful usage in a similar exposure. Mix design should contain an adequate amount of entrained air and other parameters in accordance with specifications or a special provision for a particular project. The concrete should be workable with slump and water–cement ratio as low as possible and containing at least 560 pcy (332 kg/m^3). To reduce mixing water for the same workability and, by the same token, to enhance strength and durability, a water-reducing mixture is preferred. The use of calcium chloride and type III cement for acceleration of hardening and strength development is precluded. Concrete should be handled and placed with special care to avoid segregation and prevent honeycomb and sand streaks. The proper cure should be taken for at least 7 days before exposure.

3.5.2 High-Performance Concrete

HPC is composed of the same materials used in normal concrete, but proportioned and mixed to yield a stronger, more durable product. HPC structures last much longer and suffer less damage from heavy traffic and climatic condition than those made with conventional concrete. To promote the use of HPC in highway structures in the United States, a group of concrete experts representing the state DOTs, academia, the highway industry, and the Federal Highway Administration (FHWA) has developed a working definition of HPC (FHWA 2005), which includes performance criteria and the standard test to evaluate performance when specifying an HPC mixture. The designer determines what levels of strength, creep, shrinkage, elasticity, freeze/thaw durability, abrasion resistance, scaling resistance, and chloride permeability are needed. The definition specifies what test grade of HPC satisfies these requirements and what to perform to confirm that the concrete meets the grade.

An example of the mix design for the 12,000-psi high-strength concrete used in the Orange County courthouse in Florida is as follows:

The Virginia and Texas DOTs have already started using HPC, that is, ultra-high-strength concrete of 12,000–15,000 psi (80–100 MPa), in bridge construction and rehabilitation of the existing bridges.

Gradient	Weight (lb)
Cement, type1	900
Fly ash, class F	72
Silica fume	62
Natural sand	980
No. 8 granite aggregate	1780
Water	250
Water reducer	2 oz/cwt^3
Superplasticizer	35 oz/cwt^3

3.5.3 Steel

In the bridges, reinforcing bars shall be low-alloy steel deformed bars conforming to the requirements in ASTM Designations A 706/A 706M, except that deformed or plain billet steel bars conforming to the requirements in ASTM Designation A 615/A 615M, grade 40 or 60, may be used in other categories: slope and channel paving, minor structures, and so on. Prestressing steel shall be high-tensile wire conforming to the requirements in ASTM Designation A 421, including Supplement I; high-tensile seven-wire strands conforming to A 416; or uncoated high-strength steel bars conforming to A 722, including all supplementary requirements that are usually used.

In addition, epoxy-coated seven-wire prestressing steel strands in addition shall conform to ASTM Designation A 882/A 882M, including Supplement I and other requirements, for example, California Standard Specification (Caltrans 2010).

All prestressing steel needs to be protected against physical damage and rust or other results of corrosion at all times from manufacture to grouting or encasing in concrete. Prestressing steel that has physical damage at any time needs to be rejected. Prestressing steel for posttensioning that is installed in members prior to placing and curing of the concrete needs to be continuously protected against rust or other corrosion until grouted, by a corrosion inhibitor placed in the ducts or applied to the steel in the duct. The corrosion inhibitor should conform to the specific requirements. When steam curing is used, prestressing steel for posttensioning should not be installed until the stem curing is completed. All water used for flushing ducts should contain either quick lime (calcium oxide) or slaked lime (calcium hydroxide) in the amount of 0.01 kg/L. All compressed air used to blow out ducts should be oil free.

3.6 Construction Operations

3.6.1 Prestressing Methods

If steel reinforcement in reinforced concrete structures is tensioning against the concrete, the structure becomes a prestressed concrete structure. This can be accomplished by using pretensioning and post-tensioning methods (Gerwick 1997).

3.6.1.1 Pretensioning

Pretensioning is accomplished by stressing tendons, steel wires, or strands to a predetermined amount. Whereas stress is maintained in the tendons, concrete is placed in the structure. After the concrete in the structure has hardened, the tendons are released and the concrete bonded to the tendons becomes prestressed. In pretensioning techniques, hydraulic jacks and strands composed of several wires twisted around a straight center wire are widely used. Pretensioning is a major method used in manufacture of prestressed concrete in the United States. The basic principles and some of the methods currently used in the United States were borrowed from Europe, but much has been done in the United States to develop and adapt manufacturing procedures. One such adaptation uses pretensioned tendons that do not pass straight through the concrete member, but are deflected or draped into a trajectory that approximates a curve. This method is very widely used in the fabrication of precast bridge girders in the United States.

3.6.1.2 Posttensioning

A member is called posttensioned when the tendons are tensioned after the concrete has hardened and attained sufficient strength (usually 70% final strength) to withstand the prestressing force, and each end of the tendons are anchored. Figure 3.1 shows a typical posttensioning system. A common method used in the United States to prevent tendons from bonding to the concrete during placing and curing of the concrete is to encase the tendon in a mortar-tight tube or flexible metal hose before placing it in the forms.

The metal hose or tube is referred to as a sheath or duct, and remains in the structure. After the tendons have been stressed, the void between the tendons and the duct is filled with grout. The tendons become bonded to the structural concrete and protected from corrosion. A construction engineer can use prestressing very effectively to overcome excessive temporary stresses or deflections during construction, for example, using cantilevering techniques in lieu of falsework.

Prestressing is not a fixed state of stress and deformation, but is time dependent. Both concrete and steel may be deformed inelastically under continued stress. After being precompressed, concrete continues to shorten with time (creep). Loss of moisture with time also contributes to a shortening (shrinkage). To reduce prestress losses due to creep and shrinkage and to increase the level of precompression, use of not only higher-strength steel but also higher-strength concrete that has low creep, shrinkage, and thermal response is recommended. New chemical admixtures such as high-range water-reducing admixtures (superplasticizers) and slag are used for producing HPC and ultra-high-strength concrete. The new developments are targeted to producing high-strength steel that is "stabilized" against stress relaxation, which leads to a reduction of stress in tendons, thus reducing the prestress in concrete.

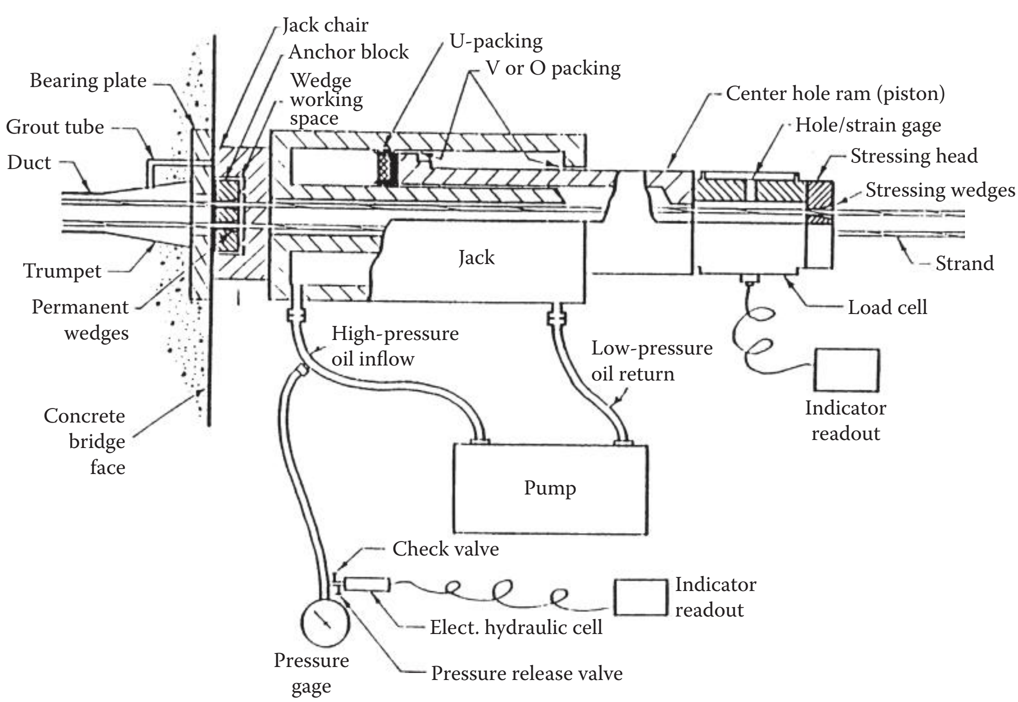

FIGURE 3.1 A typical posttensioning system.

3.6.2 Fabrication and Erection Stages

During construction, not all elements of a bridge have the same stresses that they were designed for. That is the reason it is a very important part of construction engineering to be aware of this and to make sure that appropriate steps have been taken. For example, additional reinforcement will be added to the members in the fabrication stage and delivered to the job site for erection.

In the case of cast-in-place box-girder bridge construction, the sequences of prestressing tendons have to be engineered step by step to ensure that the structure will have all parameters for future service load after completion of this stage (Caltrans 2006).

The sequence of the erection itself may produce additional stresses that structures or portions of the structures were not designed for. These stresses and the stability of structures during erection are a big concern that is often overlooked by designers and contractors—construction sequences play a very important role in the erection of a segmental type of bridge. It seems that we have to give more attention to analysis of the role of the construction engineering implementation of such erections. And, yes, sometimes the importance of construction engineering to accomplish safe and efficient fabrication and erection of bridge structures (precast, prestress girders, cast-in-pile structures) is not sufficiently emphasized by design engineers and/or fabrication contractors.

Unfortunately, we have to admit that the design drawings even for an important bridge do not include the erection scheme. And, of course, we can show many examples of misplaced erection efforts on the part of the designer, but our goal is to show why it happened and to make efforts to pay more attention to the fabrication and erection stages. Even if such an erection scheme is included in the design drawings, contractors are not supposed to rely solely on what is provided by the designer's erection plan.

Sometimes a design can be impractical, or it may not be suitable in terms of erection contractor's equipment and experience. Because the erection plans are very generalized and because not enough emphasis is given to the importance of this stage, it is important that the designer understands the contractor's proposed method so that the designer can determine if these methods are compatible with the plans, specifications, and requirements of the contract. This is the time when any differences should be resolved. The designer should also discuss any contingency plan in case the contractor has problems. In many instances, the designer is involved throughout the construction phase for design and specification clarification or interpretation, incorporation of construction changes or modifications to the project, and possible O&M reviews/action.

3.6.3 Construction of Concrete Box-Girder Bridges

A successful innovative approach in the design and construction of concrete box-girder bridges is the I-680/24 project. Built in the late 1950s, the I-680/24 interchange was expected to accommodate 70,000 vehicles a day. Today it carries an estimated 380,000 vehicles. Caltrans designed the I-680/24 to eliminate the dramatic increase in congestion and driver frustration.

It was the largest freeway interchange reconstruction project in Northern California, incorporating some of the latest seismic safety features throughout the project area, and became one of the state's most advanced freeway systems. The total cost of the I-680/24 project was $315 million.

It was built in seven phases. Phase 6 was the largest and most challenging phase of the project.

Caltrans simultaneously modified several ramps and streets serving the interchange.

The following freeway-to-freeway connectors were rebuilt to the three lanes:

- Eastbound State Route-24 to northbound I-680
- Southbound I-680 to westbound State Route-24
- Northbound and southbound I-680

The new connectors and the new on- and off-ramps throughout the project area eliminated congestion-causing traffic weaves.

Before the interchange work began, a 1 mi. temporary bypass was built in phase 3 to carry southbound I-680 traffic from Ygnacio Valley Road to Olympic Boulevard, California. This bypass begins to rise above the existing roadway at the Oakvale overcrossing, curving and descending to rejoin the permanent roadway near Olympic. When the interchange reconstruction was near completion, the bypass was removed and the materials (steel I-beams) were recycled for use in three-span bridge in the Olympic off-ramp.

Careful construction planning kept disruptions to a minimum throughout the project.

When work on phase 5 started in October 1997, Caltrans awarded a special incentive contract to R&L Brosamer, a Walnut Creek-based construction firm. The terms of the contract allow Brosamer and its subcontractors to work around the clock and in any weather to deliver scheduled work. As a result, construction activity occurred at a rapid pace.

In phase 7, Caltrans reconfigured the Monument Boulevard area into an "urban diamond interchange." The new interchange featured northbound and southbound on- and off-ramps, controlled by an intersection with one signal under the freeway instead of the traditional two intersections on either side of the street. To minimize the weave at the junction of I-680/24, exit-only lanes for Gregory Lane and Monument Boulevard from I-680 are included as part of this construction (Figure 3.2).

An innovative concrete box girder on a straight alignment over the freeway is a pedestrian bicycle bridge with a span without a median column, which crosses I-80 in Berkeley, California. The bridge is designed to handle the seismic forces generated by the San Andreas and Hayward faults, two major faults within 3–14 km of the site (Caltrans 2002).

The bridge was erected into major phase, the first being the steel-reinforced, posttensioned lightweight concrete bridge deck on falsework. Steel arches were raised and welded in place (Figures 3.3 and 3.4). Even though the bridge is primarily being used by pedestrians and bicyclists, it was designed and constructed to accommodate vehicular traffic in case of emergencies. The steel "basket handle" arch is one of the most prominent features of the bridge. Another unusual feature is the curvature of the posttensioned box-girder span.

FIGURE 3.2 Former Caltrans bridge engineer Simon Blank at the I-680/24 Interchange, Walnut Creek.

FIGURE 3.3 Berkeley Pedestrian Bridge with former Caltrans bridge engineer Hamid Kondazi, who managed the construction.

FIGURE 3.4 Berkeley Pedestrian Bridge—curved eastern approach ramp.

3.6.4 Construction of High-Performance and Ultra-High-Performance Concrete Bridges

When the first U.S. HPC bridge was built under the Strategic Highway Research Program, the FHWA and the Texas DOT sponsored a workshop to showcase HPC for bridges in Houston in 1996. The Center for Transportation Research at the University of Texas at Austin also cooperated with them at the event. It was focused on the pros and cons of using HPC; mix proportioning; structural design; HPC in precast, prestressed, and cast-in-place members; long-term performance; and HPC projects in Nebraska, New Hampshire, and Virginia. The showcase took into consideration local differences in cements, aggregates, and prestressing fabricators because they have considerable impact on the design and construction of concrete structures. In Texas, concrete can be produced with compressive strength of 13,000–15,000 psi (90–100 MPa).

The first bridge to use HPC fully in all aspects of design and construction is the Louetta Road Overpass on State Highway 249 in Houston. The project consists of two U-beam bridges carrying two adjacent lanes of traffic. The spans range from 121.5 to 135.5 ft. (37–41.3 m) long. The HPC is about twice as strong as conventional concrete and expected to have a useful life of 75–100 years, roughly double the average life of a standard bridge. A longer life span means not only lower user cost, but also motorists will encounter fewer lanes closures and other delays caused by maintenance work.

Another Texas HPC bridge located in San Angelo carries the eastbound lanes of U.S. Route 67 over the North Concho River, U.S. Route 87, and South Orient railroad. The 954 ft. (291 m) HPC I-beam bridge runs parallel to a conventional concrete bridge. This bridge presents an ideal opportunity for comparing HPC and conventional concrete.

The first spans of the two bridges are of the same length and width, making it easy to compare the cost and performance between HPC and conventional concrete. The comparison indicates that the conventional concrete lanes of the first span require seven beams with 5.6 ft. (1.7 m) spacing, whereas the HPC span require only four beams with 11 ft. (3.4 m) spacing.

To date, more than 50 HPC bridges have been built in the United States. The Jakway Park Bridge in Buchanan County, Iowa, has earned the right to be called innovative. It is the first North American highway bridge built with a new generation of ultra-high-performance concrete PI girders, the first highway bridge to incorporate UNPC batched in a ready-mix truck with compressive strength of 29,000 psi (200 MPa).

3.7 Falsework

Falsework may be defined as a temporary framework on which the permanent structure is supported during its construction. The term falsework is universally associated with the construction of cast-in-place concrete structures, particularly bridge superstructures. The falsework provides a stable platform on which the forms may be built and furnish support for the bridge superstructure.

Falsework is used in both building and bridge construction. The temporary supports used in building work are commonly referred to as "shoring." It is also important to note the difference between "formwork" and "falsework."

Formwork is used to retain plastic concrete in its desired shape until it has hardened. It is designed to resist the fluid pressure of plastic concrete and the additional pressure generated by vibrators. Because formwork does not carry dead load of concrete, it can be removed as soon as the concrete hardens. Falsework does carry the dead load of concrete, and therefore it has to remain in place until the concrete becomes self-supporting. Plywood panels on the underside of a concrete slab serve as both a formwork and a falsework member. For design, however, such panels are considered to be forms to meet all design and specification requirements applied to them.

Bridge falsework can be classified into two types: (1) conventional systems (Figure 3.5), in which the various components (beams, posts, caps, bracings, etc.) are erected individually to form the completed system; and (2) proprietary systems, in which metal components are assembled into modular units that can be stacked, one above the other, to form a series of towers that compose the vertical load-carrying members of the system.

FIGURE 3.5 Falsework at I-680/24 Interchange South-East connector, Walnut Creek.

The contractor is responsible for designing and constructing safe and adequate falsework that provides all necessary rigidity, supports all load composed, and produces the final product (structure) according to the design plans, specifications, and special provisions. It is very important also to keep in mind that approval by the owner of falsework working drawings or falsework inspection will in no way relieve the contractor of full responsibility for the falsework. In the state of California, any falsework height that exceeds 13 ft. (4 m) or any individual falsework clear span that exceeds 17 ft. (5 m) or where provision for vehicular, pedestrian, or railroad traffic through the falsework is made, the drawings have to be signed by the registered civil engineer in the state of California.

The design drawings should include details of the falsework removal operations, methods and sequences of removal, and equipment to be used. The drawings must show the size of all load-supporting members, connections and joints, and bracing systems. For box-girder structures, the drawings must show members supporting sloping exterior girders, deck overhangs, and any attached construction walkway. All design-controlling dimensions, including beam length and spacing, post locations and spacing, overall height of falsework bents, and vertical distance between connectors in diagonal bracing must be shown.

For example, in the largest freeway interchange reconstruction project in Northern California with total cost of $315 million, Simon Blank, structure engineer at Walnut Creek field office (Caltrans), reviewed all falsework drawings and calculations submitted by the general contractor, CC Meyers, Co., and then inspected falsework construction to make sure it substantially conformed to the falsework drawings. He later inspected the falsework removal operations after completion of the construction of NE and SE connectors.

As a policy consideration, minor deviations to suit field conditions or the substitutions of materials are permitted when it is evident by inspection that the change does not increase the stresses or deflections of any falsework members beyond the allowable values, nor reduce the load-carrying capacity of the overall falsework system.

But if revision is required, the approval of revised drawings by the state engineer is also required. Any change in the approved falsework design, however minor it may appear to be, has the potential to affect adversely the structural integrity of the falsework system. Therefore, before approving any changes, the engineer has to be sure that such changes will not affect the falsework system as a whole.

References

Blank, M.M. 1996. *Selected Published Articles from 1986 to 1995*, Naval Air Warfare Center, Warminster, PA.

Blank, M.M. and Blank, S.A. 1995. *Effective Construction Management Tools*, Management Division, Naval Air Warfare Center, Warminster, PA.

Blank, M.M. et al., 1997. Partnering: the key to success, *Found. Drilling*, 36(3).

Caltrans. 2002. *Transportation Journal*, March-April 2002 2(5), California Department of Transportation, Sacramento, CA.

Caltrans. 2006. *A Guide for Field Inspection of Cast-in-Place Post-Tensioned Structures*, California Department of Transportation, Sacramento, CA.

Caltrans. 2010. *Standard Specifications*, California Department of Transportation, Sacramento, CA.

FHWA. 2005. *High Performance Concrete—Structural Designers' Guide*, 1st Edition, Federal Highway Administration, Washington, DC.

Fisk, R.E. 1997. *Construction Project Administration*, Wiley, New York, NY.

Gerwick, B.C., Jr. 1997. *Construction of Prestressed Concrete Structures*, 2nd Edition, Wiley, Hoboken, NJ.

Godfrey, K.A. 1995. *Partnering in Design and Construction*, McGraw-Hill, New York, NY.

Kubai, M.T. 1994. *Engineering Quality in Construction Partnering and TQM*, McGraw-Hill, New York, NY.

Naway, E.G. 2008. *Concrete Construction Engineering Handbook*, 2nd Edition, CRC Press, Boca Raton, FL.

Schriener, J. 1995. Partnering, TQM, ADF, Low Insurance Cost, *Eng. News Rec*., January 15.

Tri-Service Committee. 1996. *Partnering Guide for Environmental Missions of the Air Force, Army and Navy*, Prepared by a Tri-Service Committee, Air Force, Army, and Navy.

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4

Cable-Supported Bridge Construction

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4.1 Introduction

The cable-supported bridges have a very broad application spanning longer obstacles and are among the oldest bridge types. The vine rope–supported bridges appeared in ancient China. The modern cable-supported bridge was born with the invention of high-strength steel. Two reasons including structural system and materials lead cable-supported bridges to longest span bridge type superior to all other bridge types including girder, arch, and truss bridges.

According to the layout shape of cables and force transfer system, the cable-supported bridges may be divided into the cable-stayed bridge and the suspension bridge.

Cable-stayed bridge is one of modern bridge types. With the increase of bridge spans and complexity of bridge structures, the construction techniques of the cable-stayed bridge become more important. The cable-stayed bridge is composed of three main components including towers or pylons, cables, and girders. The main girders of the cable-stayed bridge are connected with the tower through the inclined cables and the girder carries both the local moment and axial forces.

Construction of a cable-stayed bridge consists of four parts: foundations, piers and towers, girders, and cables. Construction of the foundation such as steel sheet pile cofferdams, steel suspension box cofferdams, caissons, and piles is similar to other type bridges. The double-wall steel box cofferdam with unique form and lots of advantages was applied in the construction of Tianxingzhou Bridge in Wuhan, Hubei province, China, and Wuhu Yangtze River Bridge, Anhui Province, China, for both railway and highway as shown in Figures 4.1 and 4.2. The construction of tower is not much different from that of pier, but the accurate location of all kinds of components is the key point because the tower is too high. The tower of a cable-stayed bridge may be built in steel or concrete. Concrete tower is mostly used in China. It is worthwhile to mention that a low tower was used in Wuhu Bridge and a new cylindrical single-column type tower in Stonecutters Bridge in Hong Kong as illustrated in Figure 4.3. The construction methods for concrete tower contain slip formwork, turnover formwork, and climbing formwork, and the main machinery contain tower crane, elevator, steel bracket, slip formwork system, turnover formwork system, climbing formwork system, and so on. For steel tower, the segments are usually fabricated in the shop and then transported to the project

FIGURE 4.1 Tianxingzhou Bridge. (Courtesy of *Wuhan Urban and Rural Construction Committee,* http://online.whjs.gov.cn/node/node_545.htm.)

FIGURE 4.2 Wuhu Bridge (http://photo.wuhu.cc/120307/25380.htm).

FIGURE 4.3 Stonecutters Bridge (http://hkbus.wikia.com/wiki).

site to be erected and installed. The segments for the steel tower are usually connected by high-strength bolts and/or welding. The segments of the steel tower may be erected by the floating crane, the climbing crane, and the tower crane. In case of inclined towers, the tower may be either installed in the horizontal state first and then rotated vertically or installed in the vertical state first and then rotated horizontally.

The girder of a cable-stayed bridge is usually prestressed concrete solid girder, steel girder, composite girders, and steel truss (Tianxingzhou Bridge and Wuhu Bridge). Similar to the construction of the girder bridges, the girders of the cable-stayed bridge may be constructed using cast-in-place cantilever, cantilever erection, casting or erection in site with support, incremental launching, swivel, floating hoisting, and so on; the concrete girder of a cable-stayed bridge is usually built by cantilever construction, but for the composite girder and steel box girder, the cantilever erection is often used during the construction.

The cables consist of three parts: anchorage at two ends, cables, and protective coating materials. The common cable materials are steel wire rope, steel bar, high-strength steel wire, and steel strand. Cable installation includes hanging cables, tensioning cables, measuring cable forces, installing energy dissipation devices, and protection. The hanging cables refer to penetrating the cables into the cable holes at the girder and the tower and fixing the cables on the anchor plate at two ends. The tensioning cables refer to adjusting the cable forces to desired conditions through jacking.

The suspension bridge is the earliest bridge type in human history (BSRI 1996). It can also span the longest distance so far in the world. The deck system of suspension bridge is carried out by the main cables suspended through towers. The modern suspension bridges are divided into two types, which are self-anchored and externally-anchored (Zhou et al. 2003). Different from the externally-anchored suspension bridge, the main cables of a self-anchored suspension bridge are anchored on the stiffening girders directly. A self-anchored suspension bridge is constructed by a sequence of main towers, girders, main cables, and suspender cables. A ground anchor suspension bridge is built by a sequence of main towers, anchorages, main cables, suspender cables, and main girders. In China, many long suspension bridges are the ground anchor suspension bridges including Xihoumen Bridge, Runyang Bridge, Jiangyin Bridge, and Yangluo Bridge as shown in Figures 4.4 through 4.6. The super large caisson foundation was used in the anchorage of Jiangyin Bridge and the deep anchor foundations in that of others.

FIGURE 4.4 Runyang Bridge (http://civil.seu.edu.cn/s/321/t/1688/a5/2f/info42287.htm).

FIGURE 4.5 Jiangyin Bridge (http://www.cots.com.cn/JingDian/JiangYinChangJiangDaQiao.html).

FIGURE 4.6 Yangluo Bridge. (Wuhan Society of Traffic and Transportation, http://www.whjt.gov.cn/jtxh2011/list.asp?articleID = 30844.)

The construction methods of main towers of a suspension bridge are similar to that of a cable-stayed bridge. For the reinforced concrete tower, the turnover formwork and the slip formwork are often used, and in addition, the stiff skeleton need usually be distributed in the construction of inclined tower pillar. For the steel towers, the prefabricated tower segments including very heavy cable saddle are erected and assembled in project sites. The cable saddle consists of saddle grooves, saddle seats, and pedestals. The saddle groove is usually made of cast steel and the saddle seat is welded with steel plates, and thus, the saddle body may be welded with the saddle seat and the saddle groove. To adjust the relative displacement, the Teflon plate is placed between the saddle body and the pedestal. The saddle body is often fabricated into two halves and assembled using high-strength bolts. The saddle's displacement relative to the pedestal is controlled with the aid of the temporary jacks set in the top of tower. The prestressed concrete transverse beams of the tower may be cast in place or precast. The anchorage carrying most of the loads of deck systems transferred by the main cables is a very important part. Gravity anchorage is usually made of reinforce concrete. The anchor system with anchor rods and beams shall be installed accurately in accordance with the design plans and specifications. Temperature shall be controlled strictly to avoid cracking during casting the concrete.

After completing the construction of main towers and anchorages, the construction of main cable may start. Since the cable is the main load-carrying component, its performances of anticorrosion and antifatigue determine the service life of bridge directly. Two methods, prefabricated parallel wire strands and aerial spinning, are often used for the bridge cable construction.

In this chapter, unique construction features of several Chinese cable-supported bridges will be presented in detail. Chapter 2 discussed general construction considerations of steel bridges. Chapters 10 and 11 provide comprehensive design considerations for cable-stayed bridges and suspension bridges, respectively.

4.2 Cable-Stayed Bridges

4.2.1 Foundation Construction

4.2.1.1 Double-Wall Steel Cofferdam

4.2.1.1.1 Construction Features

The double-wall steel box cofferdams, as a new construction scheme in foundation construction, have been successfully used for cable-stayed bridges in China (Luo and Yu 2002; Luo 2003). Its unique features and distinct merits are summarized in the following discussion.

Feasibility The construction of double-wall steel hoisting box cofferdam uses the buoyancy provided by the compartments of double-wall side plates and bottom plates. Due to full use of the waterway conditions and floating haul techniques, major steel structures such as steel cofferdam, drilling platforms, and steel orientation guide frame can be fabricated and assembled in the factory on riverbanks rather than assembled under water. It not only reduces the field operations and shortens the construction period but also ensures the accuracy of the steel components and the steel casing tubes.

Double-Wall Side Plates Compartment Structure The steel box cofferdam with double-wall side plate compartments use the buoyancy provided by the hydraulic head difference between inside and outside the cofferdam. It meets the drilling construction requirement at different construction water levels without increasing the height of the cofferdam. The whole cofferdam including pile caps may be sunk as long as water or concrete is poured into the double-wall plate compartments. In case of construction of pile caps of deep water piles, it reduces the height of cofferdam significantly and saves lots of steel materials.

Bottom Compartment Structure In the steel box cofferdam, the bottom compartments are used so that several small rooms may be formed, which is favorable for casting the bottom sealed concrete. As guided by the inner frame, the cofferdam is hung on the steel positioning tubes and then bored pile construction may begin. After completion of the bored pile construction, the anchor system of the cofferdam anchor pier should be restored and the cofferdam is released from the corbel of the positioning tubes for the first time and then water is poured into the double-wall side plate compartments to sink the cofferdam to the design elevation. It is followed by (1) hanging the cofferdam on the positioning tubes for the second time, (2) pouring the bottom sealed concrete, (3) pumping water out of the cofferdam after the sealed concrete gets to the design strength, (4) cutting off the steel positioning tubes and suspender above the sealed concrete, and (5) placing reinforcement and pouring mass concrete for pile cap.

4.2.1.1.2 Construction Difficulties

Due to the unique double-wall structure of steel cofferdam, major difficulties are given in the following discussion.

- Bored Piles
  Because of the large dimension, structure complexity, big current resistance of rectangle cofferdam structure, the fabrication, assembly, floating, and positioning, cofferdam construction is very difficult. The construction site is often located near the main busy navigable span, which will affect the steel cofferdam floating location and foundation construction with high-safety risk. Therefore, it is important to ensure that enough bored piles are constructed to form the stable construction plate form before the flood season.
- Anchor Positioning
  The steel cofferdam, double-wall self-floating structure is mainly used to construct the pile cap and provide a platform for bored pile construction. Therefore, it is very important to anchor the cofferdam in an accurate position. There are two issues that need to be solved at the time of the anchor positioning to keep stresses in each anchor as uniform as possible and to properly adjust anchor due to changeable want levels.

Basic features and difficulties of double-wall steel box cofferdam (Li and D 2010; Li and Qin 2006) are discussed in two representative cable-stayed bridges built recently in China.

4.2.1.1.3 Bridge Projects

Tianxingzhou Bridge Tianxingzhou Bridge (Li and Qin 2006) is the sixth bridge and the second bridge for highway and railway across the Yangtze River at Wuhan, China. The bridge construction began on September 28, 2004, and was opened to traffic on December 26, 2009. Tianxingzhou Bridge is the biggest bridge for highway and railway with the total investment of about 11.06 billion RMB in China. A double-wall steel box cofferdam was used to construct Piers 2 and 3. Table 4.1 summarizes the basic technique features of the Tianxingzhou Bridge cofferdam (Qin et al. 2007).

The cofferdam of Pier 2 was fabricated segmentally in the factory and assembled on the banks. The double walls were divided into two segments. The 6.5-m lower segment of 4.6-m high was moved to the river using air bags. After the lower segment floated in the river, the top segment of the lower section was assembled on the river. And then, the whole floating structure was moved to the project site using a main towboat and an aid towboat.

The cofferdam of Pier 3 was fabricated segmentally in the factory and assembled as a whole on the ship platform. The cofferdam was moved into the river using the rails of inclined ship supporting frame. The cofferdam floated to the location using two main towboats and an aid towboat as shown in Figure 4.7.

Compared with a traditional steel cofferdam assembled at the pier location, the advantages of the overall floating method are higher quality of steel cofferdam fabricated in the shop, accelerated the construction progress due to the synchronous operation with the anchor construction at the pier location, reduced field operations such as assemblies and welds, and use of smaller crane boats and the sinking system.

Movable Cofferdam Fixed by Piles Technique The water level for 20-year flood is 27.32 m at the bridge location and the design water level was selected as 26 m. The bottom elevation of Piers 2 and 3 pile cap is 4 m, the thicknesses of bottom sealed concrete are 2.5 m and 3 m, and the bottom elevations of cofferdam are 1.5 m and 1 m, respectively. In case of conventional cofferdam construction positioned by piles to the desired location only one time, the top elevation of the cofferdam is 26 m and its total height is 25 m, which may increase 1000 tons steel in the cofferdam construction for Pier 3; the biggest water level difference is 12 m and the top of the cofferdam is 14 m above the water level in dry season. It is very difficult to construct. Using the characteristics of the cofferdam's floating body, the cofferdam is designed as a water-resisting structure and a floating platform with the inner supporting frame to form a drilling platform and positioning piles platform. The double-wall steel cofferdam of Piers 2 and 3 are 14.5 and 15 m high without including the height of bottom plate, and plus a 5 m high single wall to ensure the piers can be constructed above the water level before the

TABLE 4.1 Basic Features of Cofferdam of the Tianxingzhou Bridge

Structure size of main pier cofferdam	57.6 × 31.2 × 20.1 m (Pier 2)
	69.5 × 44.0 × 20.6 m (Pier 3)
Height of bottom plate	0.6 m (Pier 2)
	0.6 m (Pier 3)
Height of double-wall side plate	14. 5 m (Pier 2)
	15 m (Pier 3)
Height of single-wall side plate	5 m (Pier 2)
	5 m (Pier 3)
Thickness of double wall	2 m (Pier 2)
	2 m (Pier 3)
Self-weight of steel structure (not including single wall)	2100 ton (Pier 2)
	3100 ton (Pier 3)

flood season. Based on the concept of combination of single wall and double walls for cofferdam, the cofferdam met not only force demands of the weight of bottom sealed concrete and construction loads of pumping and drilling but also demands of resisting water in the dry season. The height of the single wall may be adjusted according to the construction schedule and the water level. Despite increasing several working procedures, the height of cofferdam was shortened significantly and the cost was saved tremendously.

The cofferdams of Piers 2 and 3 floated mainly by self-buoyancy, aided by the reaction provided by the corbel set on the positioning steel pipe piles, and therefore, there is no need to remove the large machineries on the cofferdam. With designed steel positioning tubes as illustrated in Figure 4.8, the cofferdam can be positioned and fixed by positioning steel pipe piles in a short time.

Anchored Pier Positioning Technique The cofferdam of Pier 2 was fixed by the anchored pier (steel pipe piles) positioning technique (Yuan 2008). The cofferdams are positioned accurately by stretching prestressed steel strand because the stiffness of prestressing system is much bigger than that of the regular anchor chain plus the steel wire rope. The positioning system of the anchored pier consists of an anchored pier of steel pipe piles foundation, prestressing system, and a reinforced concrete pile cap to increase stiffness of the steel pipe piles and to ensure a proper load-carrying path. After the cofferdam was floated to the location and fixed with the steel strand on the anchored piers, the steel strands were

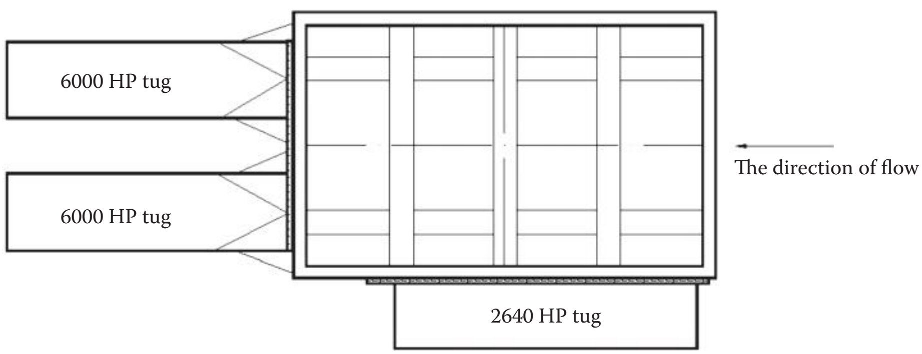

FIGURE 4.7 Floating transport of Pier 3 cofferdam. (From Li, J.T., & Qin, S.Q., *World Bridges*, 2, 17–19 2006.)

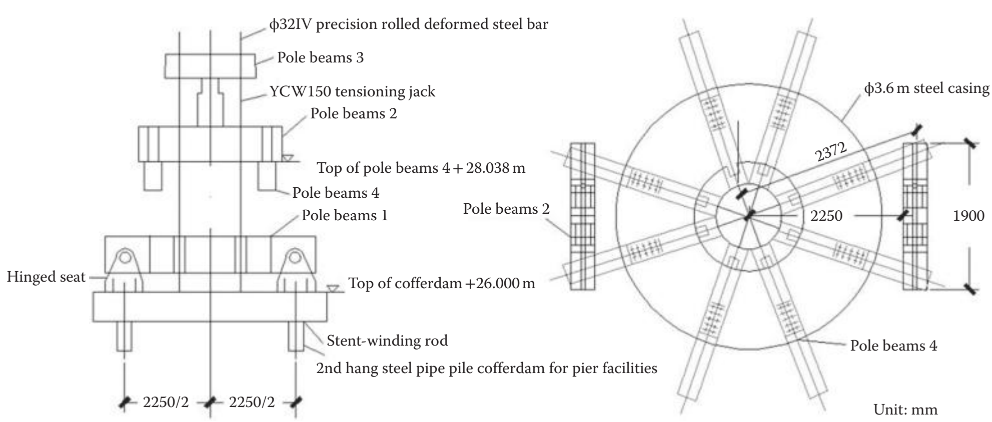

FIGURE 4.8 Fixtures of steel pipe piles. (From Li, J.T., & Qin, S.Q., *World Bridges*, 2, 17–19.)

tensioned and the tensile forces of anchored cables were adjusted to ensure the cofferdam to its designed location as illustrated in Figure 4.9.

In the anchored pier of the upstream side, there are 9 steel pipe piles of 3 rows with a 1000-mm-diameter, 12-mm-thick rolled steel plate and 2 and 3 m spaces. The slopes of batter steel pipe piles are 1:5 for outer piles and 1:11 for inner piles, respectively. The pile tip elevation is −28 m. The pile cap elevation is +23 m and the size is 8 × 6 × 2.5 m. Jacking platform was set on the pile cap and tension reaction beam was connected to the pile cap. In the anchored pier of the downstream side, there are 6 steel pipe piles of 2 rows with a 1000-mm-diameter, 12-mm-thick rolled steel plate and 2 and 3 m spaces. The slopes of batter steel pipe piles were 1:5 for the outer piles and 1:11 for the inner piles, respectively. The pile tip elevation is −28 m. The pile cap elevation is +23 m and the size is 6 × 5 × 2.5 m.

The anchored pier positioning technique is suitable for proper riverbed geology, reasonable water depth, and high accuracy positioning requirement. The positioning method for a large floating anchor was successfully used in Tianxingzhou Bridge across a great river.

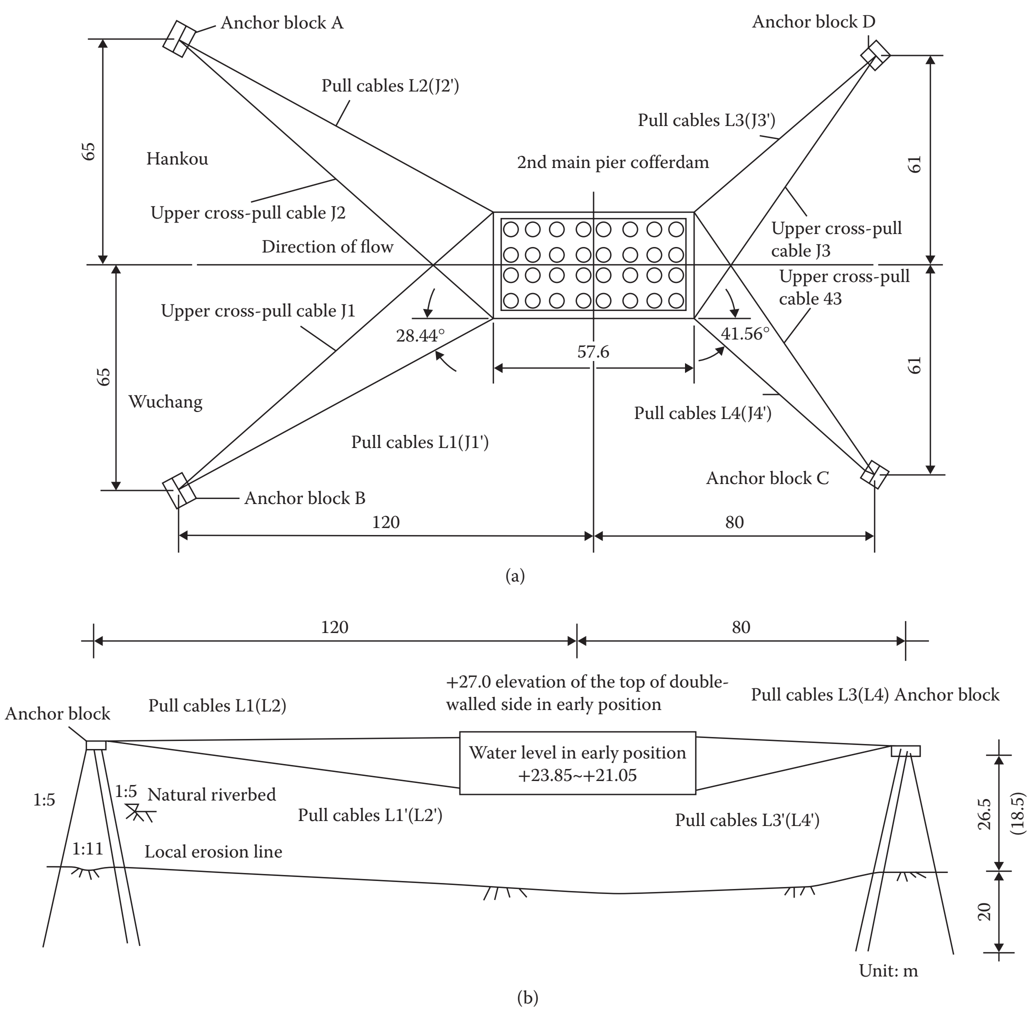

FIGURE 4.9 Positioning anchored pier. (a) Plan. (b) Side view. (From Li, J.T., & Qin, S.Q., *World Bridges*, 2, 17–19.)

General Construction Sequences Because of the closed bottom double-wall steel box cofferdam, its construction sequence is much different from those open bottom steel box cofferdams (He and Jiang 2002). The steel box cofferdam has been used in several bridges across Yangtze River, in most of which the construction platform was built first and followed by the bored piles and cofferdam construction.

If the steel box cofferdam or its bottom segment can't be erected all at once using a floating crane, much time will be spent on assembling the segments of the cofferdam, building the platform, and extending and sinking the cofferdam. It is observed that the construction time for Tianxingzhou Bridge was reduced significantly because the bored piles were constructed after floating the cofferdam.

The construction sequence time should be arranged carefully by floating the cofferdam in dry season and boring in flood season if possible.

Wuhu Bridge Wuhu Bridge (Zhou 2000a), the key transportation project in Chinese "National Ninth Five Year plan," is a steel truss cable-stayed bridge for highway and railway, class one double-line railway and four-lane highway traffic with 18-m-wide roadway and 1.5-m-wide sidewalk on two sides. The railway length is 10,616 m and the highway length 6,078 m, in which the length over the river is 2,193.7 m. The bridge construction begun on March 22, 1997, and was opened to traffic in September, 2000.

The Piers 9–128 of the main spans of Wuhu Bridge are located in the main navigation channel with deep water and over 20-m-thick overburden layer above the riverbed. The foundation was designed as a circular double-wall steel cofferdam of 22.8 m × 30.5 m with 2.8 ~ 3.0-m-diameter bored piles. The steel cofferdam is 36.2 ~ 53.2 m high and weighs 454 ~ 880 tons. The major cofferdam construction innovations of the Wuhu Bridge are manufacture, floating transportation, sinking landing riverbed, and cleaning foundation.

Manufacture and Floating of the Steel Cofferdam The steel cofferdam was fabricated segment by segment on a horizontal assembling platform in the shipyard. The manufacturing and assembling sequences were exterior wall plates, stiffening steel angles, horizontal supports, diaphragm plates, stiffening steel angles of interior wall plates, interior wall plates, demoulding, turning over, and welding.

The steel cofferdam was first dragged inside two guiding ships. As shown in Figure 4.10, the steel cofferdam was placed in the middle of the guiding ships using four flexible cables suspended on the aisle beams and balanced by 5 ton weight with pulley blocks. Additionally, the front and rear positioning ships were connected by the cables and anchored in 200 and 3 m upstream of the pier location, respectively.

Sinking Cofferdam to Riverbed The sinking of the cofferdam with the accurate position and proper vertical straightness down to riverbed was a key for construction success. The sinking of the cofferdam to the riverbed is a process from its suspended floating state to the relative stable state after embedded in riverbed to some depth. Considering the characteristics of the regional climate of the bridge, the steel cofferdam was sunk to riverbed during the dry season when the current speed of Yangtze River was low and the riverbed scour was not severe. The sinking of the cofferdam was controlled easily and the riverbed was leveled easily. The technique features are summarized in the following discussion.

Equipment The sinking of the cofferdam to the riverbed depends on the weight of injecting water into compartments before the landing the riverbed. The cofferdam sunk by both injecting water and suctioning mud after parts of the foot blades embedded into the riverbed. Therefore, the three types of

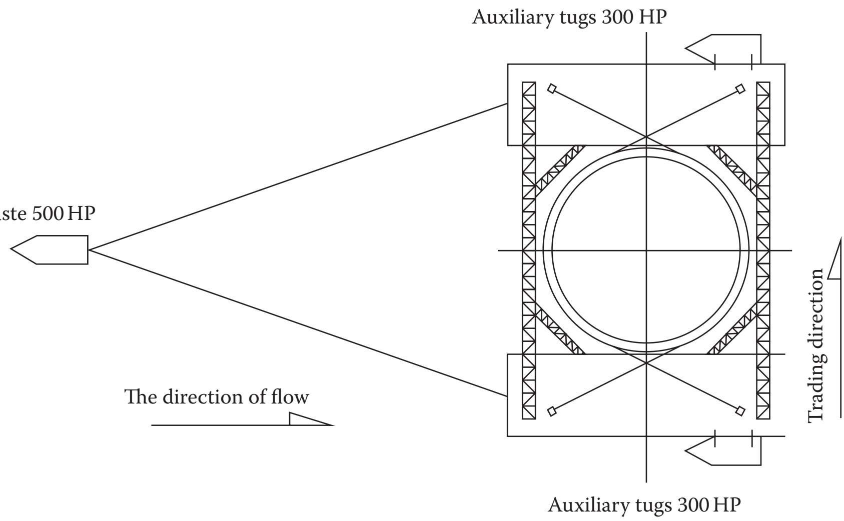

FIGURE 4.10 Floating transport of steel cofferdam, Wuhu Bridge (Wuhu Yangtze River Bridge double-wall steel cofferdam construction).

equipment, water pumps, mud suction pumps, and air blowers, were used to ensure enough water into the compartments and proper air supplies.

Cofferdam Location To stabilize the positioning ships at the center of pier, a number of cables were set to adjust the location during the landing of cofferdam to riverbed.

Elevation Measurement of Riverbed The elevations of riverbed at the location of the foot blades were measured at regular intervals and the scour effect induced by the sinking of the cofferdam was analyzed to determine whether the height of the cofferdam should be increased since the cofferdam floated to the location of piers.

Leveling Riverbed Leveling the riverbed is very important for the smooth landing of the cofferdam on the riverbed and the proper mud suction sinking. It is necessary to level and control the elevation height difference within 10%.

Once the foot blade of the cofferdam cut over the riverbed, the measurements were conducted immediately to determine the range of reaching riverbed. Since the waterhead difference between inside and outside compartments was not allowed to exceed the allowable design value, the cofferdam was still partially suspended in water. The cofferdam could not continue to sink but swung and inclined due to water wave impacts. Therefore, the mud suction should proceed at once at the higher elevations of riverbed. When the cofferdam embedded partially in a higher elevation of the riverbed downstream and blocked the current, parts of the cofferdam upstream and side were still suspended as illustrated in Figure 4.11. The sediments flowing in upstream accumulated in the cofferdam and therefore the riverbed at the location of the foot blade was elevated. The cofferdam would be more stable because of the support from different directions as illustrated in Figure 4.12. With the flow diminishing from the side, the side scour was influenced and had an elevating trend. After a series of cyclic water injection and mud suction, the cofferdam was sunk slowly and formed a closed system. The system was sunk by the regular mud suction method and leveling the waterheads in compartments after a closed system was formed. In case of larger inclination and displacement when the cofferdam lands on the riverbed, the cofferdam has to be floated and landed on the riverbed again.

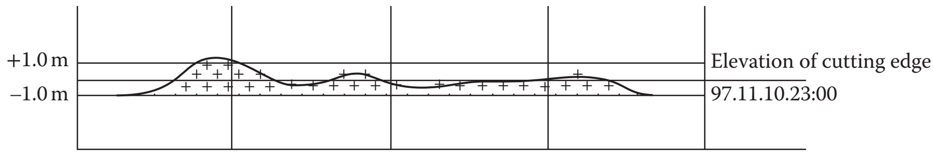

FIGURE 4.11 Elevation one—foot blade and riverbed (Wuhu Yangtze River Bridge double-wall steel cofferdam construction).

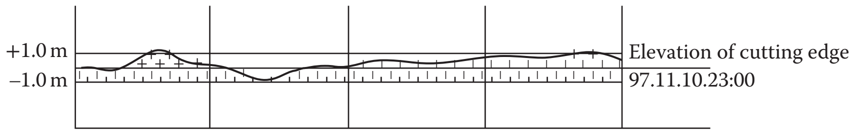

FIGURE 4.12 Elevation two—foot blade and riverbed (Wuhu Yangtze River Bridge double-wall steel cofferdam construction).

Sinking Cofferdam to Bedrock Sinking The sinking of cofferdam to the bedrock should follow the following principles:

1. The inclination and the displacement appeared during sinking and outgassing should be adjusted by the displacement first and then the inclination. The purpose of displacement adjustment is to make proper inclination for sinking. The inclination angle should be controlled within 1%.
2. To guide the mud suction and avoid sand overturning, the mud elevation within the cofferdam should be measured carefully and timely.
3. In the early stage of sinking, supplemental water should be pumped into the cofferdam to ensure that the water elevation inside the cofferdam is not lower than outside the cofferdam.
4. In sinking the overburden mud layers, the weight of cofferdam should be larger than the friction forces and the sinking coefficient should not be less than 1.25. Therefore, the underwater concrete should be poured several times to sink the cofferdam smoothly and enhance the strength of cofferdam to resist the bigger waterhead.
5. The elevations of riverbed should be measured periodically when sinking the cofferdam. The elevation difference around the cofferdam should be controlled within 3 m based on the location of cofferdam center. In case of larger elevation difference, protective measures such as dropping sand bags and flagstones or mud suction at higher locations should be taken.

Clearing Foundation Clearing foundation may start when the cofferdam is sunk about 1 m above the bedrock. Clearing procedure is as follows:

1. Using high pressure water jets and mud suction pumps in the area of the cofferdam 3 to 5 m away from the wall, something like mud, clay, gravel, and strong weathered rocks is cleared thoroughly out of the cofferdam.
2. By using mud suction pumps at the interior wall of cofferdam close to the higher position of bedrock, the foot blade may land on the higher position of the bedrock surface. With high-pressure water jetting and mud suction pumps, the strong weathered rocks may be exfoliated to continue sinking the cofferdam.
3. Once the foot blade touches the bedrock, the mud suction pumps and high-pressure water jets should be moved from one side to the other along the interior wall.

With 50-m waterhead difference, the double-wall steel cofferdam was strong enough for the smooth construction of deep water foundations and the part above sidewall concrete may be taken advantage of and recycled. Due to its low water floating and landing in the riverbed during dry season, Wuhu Bridge becomes a classic case in deep water foundation constructions of cable-stayed bridges.

4.2.1.2 Thin-Wall Wide Pier Construction

The Tianxingzhou Bridge is the first bridge using the thin-wall wide pier construction technique in China. The total length of Tianxingzhou Bridge is 1092 m long, spanning to the mainstream of Yangtze River, located in Wuhan, Hubei Province. The main bridge is a cable-stayed bridge with double towers plus three cable planes and 98 + 196 + 504 + 196 + 98-m span arrangement. The bridge approach in Piers 0–28 of the north bank is for both highway and railway with two passenger lines and two freight lines. The pier is 25.7 ~ 37.2 m long, 4.8 m wide, and 23.2 ~ 37.2 m high. The 2-m bottom segment and 3-m top segment of piers are solid, and the remaining parts are thin wall with 1-m-thick box section. The new construction techniques are summarized in the following discussion.

4.2.1.2.1 Main Construction Features of Tianxingzhou Bridge Piers

Selecting Concrete Material and Mix Ratio The lower hydration heat Portland cement was used in the pier concrete, mixed with early strength and retarding function additive FDN-5 (High Efficient Water-Reducing Admixture) and durable and qualified gradation gravel aggregates. Medium size sand of modulus 2.7 was used to guarantee better workability. To reduce the hydration heat, prolong the initial set of concrete and delay the time of hydration heat peak appearance; fly ashes in place of cement and high-efficient-water reducing additive, also called as "double-mixture technique," were also added to the mix ratio of pier concrete, which are listed in Table 4.2.

Shortening Time Difference between Pile Cap and Pier Bottom The concrete shrinkage at different ages is different. To minimize the shrinkage difference between the pile cap and the pier bottom segment to reduce tensile stress in concrete, concrete pouring time for the pile cap and the pier bottom segment was carefully arranged.

Arranging Circulating Water Pipes To reduce the interior and exterior temperature differences during concrete hardening, the circulating cooling steel water pipes, 48 mm diameter and 3.2-mm-thick wall spaced 1 m, were placed in the pier bottom segment. During concrete casting, water started to flow in cooling pipes as soon as concrete covers the cooling pipes, and the temperature difference between the inlet and outlet water was controlled to 6°C. To avoid the temperature difference exceeding 6°C, pressure water pumps were installed at the inlet to increase water flow and velocity so that hydration heat in the concrete may be avoided.

Adding Mesh Reinforcement for Anticracking To prevent pier cracking, one layer of 10 × 10-cm mesh reinforcement of 3-mm-diameter wires for anticracking was placed with 3 cm clearance within 5 m range above the bottom of the pier.

Applying "Formtex" Permeable Formwork To improve concrete surface strength and density in the early stages, a layer of "Formtex" permeable formwork cloth was stuck on the interior surface of the formwork. After vibrating, concrete surface water can be removed through capillary fiber of the permeable formwork cloth. This method played a significant role in controlling the cracking of concrete surface.

TABLE 4.2 Concrete Mix Ratio of Pier Concrete (kg/m^3)

Cement	Fly Ash	Sand	Gravel	Water	FDN-5
255	85	756	1134	170	2.72

Controlling Temperature in Mass Concrete To ensure the temperature of pouring concrete below 30°C in summer, cooling water measures were used in mixing concrete. The cooling water tower was installed to control the temperature of water used in the north approach spans of Tianxingzhou Bridge. Since the temperature of water in concrete mixture was required not to exceed 15°C, the sun sheds were installed covering the aggregates to avoid direct sunlight and the aggregates were washed with cooling water. This approach guaranteed the concrete was mixed and controlled within the specified temperature.

Postponing Formwork Removal Time Earlier removal of the exterior formwork may cause loss of humidity and surface shrinkage cracking. Therefore, the time of exterior formwork removal was postponed and usually controlled within 7 days to ensure the shrinkage cracking of concrete was controlled.

Installing Curing Pipe around Pier Considering the humid hardening environment of piers, the concrete volume during hardening does not shrink but expand. Therefore, during the construction of thin-wall wide piers of the north approach spans, curing pipes were installed immediately around the pier after the formwork was removed. The 5-cm diameter PVC curing pipes have water holes of 3 mm diameter spaced 10 cm along pier height. A 10-m^3 storage water tank was set beside the pier and the inlet water was conveyed directly by the high-pressure water pump. Water was sprayed evenly on the outer surface of the pier through the water holes of the PVC pipe, which can guarantee that the whole outer surface of the pier was in the humid environment suitable to concrete hardening. The temperature stress caused by concrete hydration heat was, therefore, reduced and the shrinkage cracking was avoided.

Arranging Concrete Pouring Position To ensure the construction quality and arrange the pouring points of concrete rationally, the pouring points were arranged as a quincuncial layout and the pouring height was adjusted within 50 cm. This arrangement may prevent concrete from segregation while pouring.

4.2.1.2.2 Concrete Crack Treatment

Cracking of pier was significantly reduced and even avoided by adopting the preceding measures. Concrete quality of piers satisfied the Chinese Specifications (CNMOR 2005). However, vertical cracks about 0.15 mm in width do appear near the centerline of several piers. Although the width of the cracks meets the requirement of the Chinese Specification, the cracks were repaired to enhance the concrete durability.

Xypex material, which has the features such as permeability, enhancement, durability, and nonpolluted, was introduced from Canada. The Xypex mortar was painted on the surface of the cleaned concrete, which can produce catalytic crystallization reaction in the concrete surface pores with moisture, resulting in insoluble dendrite-like crystals. The crystals not only protect the outer surface of concrete but also fill the concrete surface pores with crystals. Taking the preceding measures, the thin-wall wide pier construction of the Tianxingzhou Bridge becomes a good example of cable-stayed bridges.

4.2.2 Tower Construction

4.2.2.1 Low Tower Construction

This section presents innovative construction techniques developed in the low tower construction of the Wuhu Bridge.

The Wuhu Bridge has total railway length of 10,520.97 m and highway length of 5,681.20 m. The main bridge is 2,193.7 m long with a main cable-stayed bridge span of 312 m.

To meet both the aviation and navigation requirements, the main span was designed as a steel truss low tower cable-stayed bridge. There are lots of vertical prestressing facilities and many horizontal and longitudinal prestressed steel installed in anchored areas to accommodate complex stress states in the main towers. Construction of low tower is more difficult than the construction of typical high towers. The main tower consists of lower tower columns, middle tower columns, crossbeams, upper tower columns, and anchored areas. The construction layout of the main tower is shown in Figure 4.13.

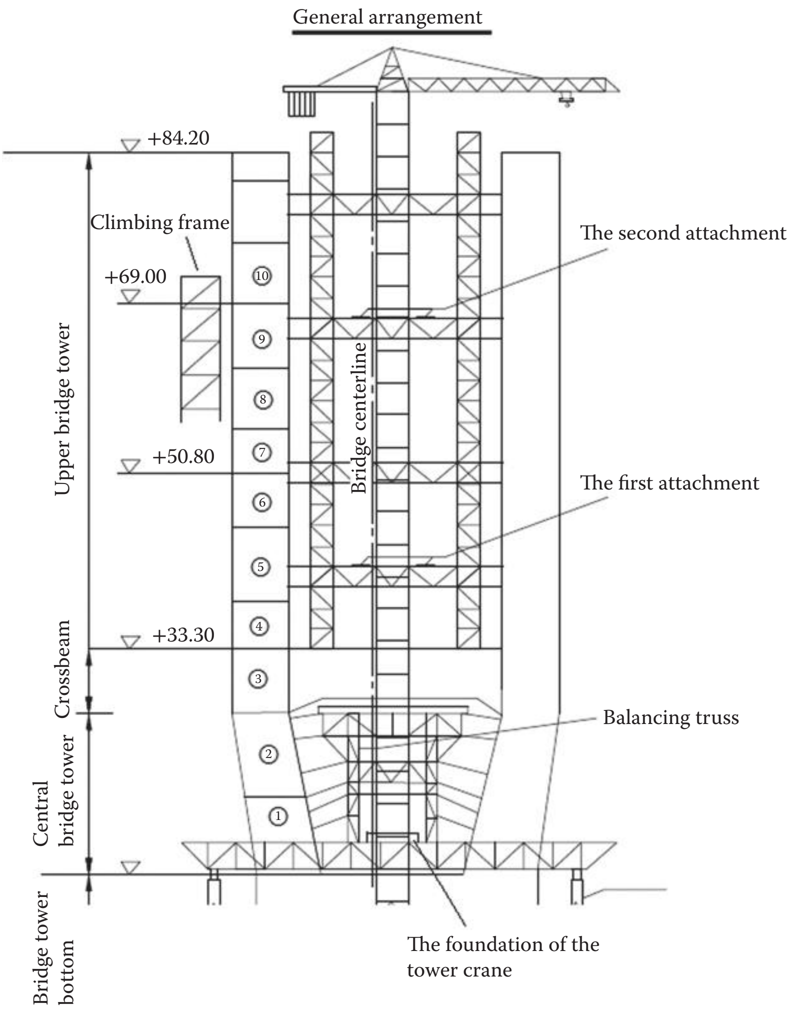

FIGURE 4.13 Construction layout of main tower pier. (From Zhou, M. B., *Bridge Construction*, (2): 14–19 (in Chinese), 2000a.)

The bottom and top segments of the low tower column are about 6 m. Concrete of Grade C50 was continuously cast for every segment of 1790 m^3. To prevent the mass concrete from cracking due to hydration heat, some cooling temperature measures such as adding fly ashes and cooling pipes were used. A scaffolding method was used in the construction of crossbeam preloaded with 1000 ton weight to remove the influence of deformation. Micro-expansive concrete was used for crossbeam to avoid/reduce shrinkage cracks.

The upper tower column and anchored area were casted segmentally using the climbing formwork platform, a 1400 kN tower crane, and a concrete pump. The segment height is 6 m.

Eight pairs of cables were installed on each side of the towers, which is 33 m high from the highway bridge deck with a height-span ratio of 1/9.5, far less than one-fourth of typical Chinese cable-stayed bridges. The lower part of the tower has a two-cell single box section at elevation below 12 m, two inclined legs of a single-cell box section at elevations between 12 and 33.3 m with the crossbeam forming a close frame. The upper tower has two sole columns of a single-cell box section that is 8.5 × 4.4 m. The anchored area is a single-cell section with a 2.1-m-thick anchored wall and 0.9-m bearing tension wall. The cables are anchored in inner grooves and the center distance between the anchor points is 1.2 m. A thicker steel bearing plate, spiral reinforcements, and shearing rings were placed under the anchor. The anchored area was designed as a partially prestressed concrete. Deformed high-strength bars and high-strength wire strands were used in anchored walls and longer bearing tension wall, respectively. Vertical prestressed steel strands were placed in the area range between 50.8 m (highway deck) and 12.0 m (bottom of inclined legs) elevations.

Due to the structure system of the Wuhu Bridge, the stress in the main tower is complex. Since there were dense steel bars, stiff skeletons, vertical prestressed steels, horizontal prestressed steels, cable ducts, and anchor reinforcements, the main tower was very difficult to construct. The Wuhu Bridge is a milestone of low tower cable-stayed bridge construction technique in China.

4.2.2.2 Cylindrical Column Tower Construction

The most famous bridge with a cylindrical single column tower is the Stonecutters Bridge in Hong Kong (Gong 2004; Huang and Xu 2007). The bridge has a main span of 1018 m and anchor span of both the sides is 289 m, consisting of four spans of 79.75 + 2 × 70.00 + 69.25 m (Xu and Huang 2004). The least height of navigation clearance is 73.5 m. The tower materials are concrete from the bottom to elevation 175 m and steel–concrete composite from 175 to 293 m, and the outside surface is made of stainless steel.

4.2.2.2.1 Tower Foundation

There are 27 and 29 bored piles with 2.8 m diameter in the east and the west tower foundations, respectively. Piles depths are from 60 to 110 m. The size of concrete pile cap is 47.4 × 36.4 × 8 m. The key measures of building a big pile cap alongside sea embankment are pumping water and temperature control during pouring concrete as illustrated in Figure 4.14. The builder installed 20 water pumps with 200-mm-diameter pipes inside the sheet pile cofferdam and set a 24-hour automatic level monitoring system in several important places to ensure that the settlements of the nearby building are not excessive during the foundation construction. Through a series of measures such as the application of concrete

FIGURE 4.14 Tower foundation construction, Stonecutters Bridge (http://www.zgjlw.net).

containing 60% ground granulated blast furnace slag, adding ice to the concrete, and limiting thickness of poured concrete to 1 m, the highest temperature of pouring concrete may be controlled within 80°C and the maximum temperature difference inside concrete should be less than 3°C.

4.2.2.2.2 Tower

Stonecutters Bridge is supported by two near 300-m high cylindrical towers. The lower part of the tower was constructed by the climbing formwork method and the upper part's stainless steel hulls as the tower surface and permanent formwork was installed by a tower crane. The stainless steel hull segments were connected using high-strength bolts.

From the beginning of Chinese infrastructure development, bridge construction was just to meet the transportation demands. But today, engineers have to consider bridge aesthetics, environment impacts, and life quality. The future bridges will be designed as a more practical and more aesthetic development direction. The Stonecutters Bridge reflects a creative spirit of people in Hong Kong and perfectly suitable for an international metropolitan like Hong Kong.

4.2.3 Cable Construction

Cables are an important part and its construction influences the quality and service life of cable-stayed bridge directly. In the construction of the Tianxingzhou Bridge, it took 5 months to install 4000 ton cables (Zuo and Zhang 2009), and the features of the cables construction are summarized in the following discussion.

4.2.3.1 Construction Preparation

Preparation works of cable construction include traction force calculation of the anchor installation and equipment configuration. The traction force of cable anchor installation was calculated according to the cable force provided in the design plans. Considering the final design cable forces, 1200- and 1500-ton tensioning jacks were used for side spans and the main span, respectively. The traction equipment at the end of girder contains 250-ton jacks and relative brackets. Other equipment contain 10-ton hoisters with 500 and 750 m rope capacity, floating crane, auto-crane, tower crane, and so on.

4.2.3.2 Cable Installation

The procedures of the cable installation are transportation, boarding, hoisting cables up to the bridge, hanging cables at the end, spreading cables, tensioning cables within the tower, checking cable forces, adjusting the cable forces, and so on.

4.2.4 Main Girder Construction

4.2.4.1 Composite Girder Construction

This section presents unique features of the composite girder construction in the Wuhu Bridge (Ye and Hou 1999; Li et al. 2001).

4.2.4.1.1 Deck Slab of Composite Girder Construction

The deck slab of the Wuhu Bridge is a unique load-carrying structure system (Yang 2007). Prefabricated highway deck slab is composited with the top chords of the steel truss using shear studs.

The deck slabs of the cable-stayed bridge span are divided into five segments along the transverse direction with the largest size being 4.07 × 11.52 m. The deck slabs of continuous girder spans are divided into four segments along the transverse direction with the largest size being 5.625 × 11.25 m. Due to the larger area, relative thin thickness, and shear keys around all sides, the lap rebars were placed within 3 mm average error to meet high installation requirements of the shear studs.

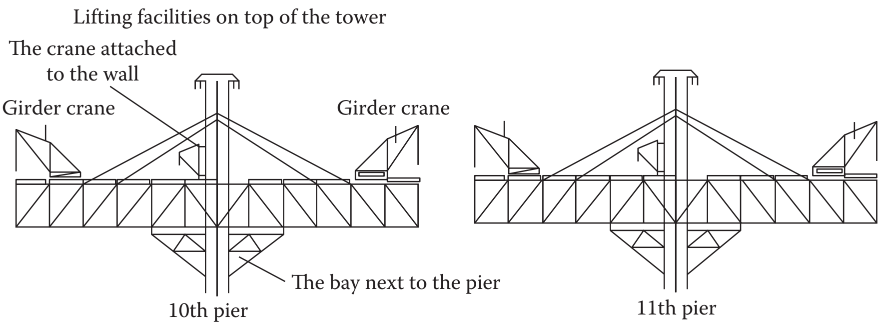

FIGURE 4.15 Steel girder erection. (From Zhou, M. B., *Bridge Construction*, (2): 14–19 (in Chinese), 2000a.)

The deck slab segments in the same panel were prefabricated in the specifically designed platform according to their relative positions in the final stage. The platform was supported by square pile foundation to prevent desk slab cracking from uneven settlements.

The deck slabs with prefabricated lifting holes were moved and loaded by a 500-kN gantry crane. The deck slabs within a single internode were usually stacked horizontally together. Keep steady when the deck slabs are transported and stored to avoid deck slab cracking. For convenience of the installation on the bridge deck, an assembling mock platform was installed on the project site to simulate the assembly of steel girder shear studs. It made the bridge deck slab erection and assembly much easier and fast.

4.2.4.1.2 Composite Girder Erection

Cantilever Girder Erection The method of steel girder erection is innovative (Zhao and Li 2001). During construction, the girders were erected by cantilever construction method from the main tower Piers 10 and 11 to their two sides as illustrated in Figure 4.15.

Main Span Closure The main span closure of the cable-stayed bridge is a key procedure during the girder erection. Due to the longer closure length and the complex structure system, two-time closure technique, temporary hinges closure of top and bottom chords firstly and then closure of the truss diagonals, was used successfully.

4.2.4.2 Steel Truss Girder Segment Installation

In China, the segments of steel truss girder for cable-stayed bridge are usually prefabricated and assembled into a whole structure in the factory. The whole truss structure is then transported and erected in the construction site. In this section, the innovative construction features of the steel girder erection of the Tianxingzhou Bridge will be presented as an example of the steel truss segments installation.

Firstly, four interpanel truss segments over the pier top were assembled and then erected by the girder erection crane. With the success of one interpanel truss segment installation, the girder erection crane moved forward one interpanel length, after hanging the cables for the previous truss segment, installing the orthotropic plates and tensioning the cables, and then internode girder erection was continued. When the steel truss girders of two main towers were very close, the mid-span closure truss segment was erected firstly and then the segments without cables of side spans were assembled continuously with cantilever style.

4.2.4.2.1 Interpanel Steel Truss Segment Erection on Pier Top

Using falsework installed on sides of the pier as an operation platform, the interpanel steel truss segments are assembled and erected by a floating crane as illustrated in Figure 4.16.

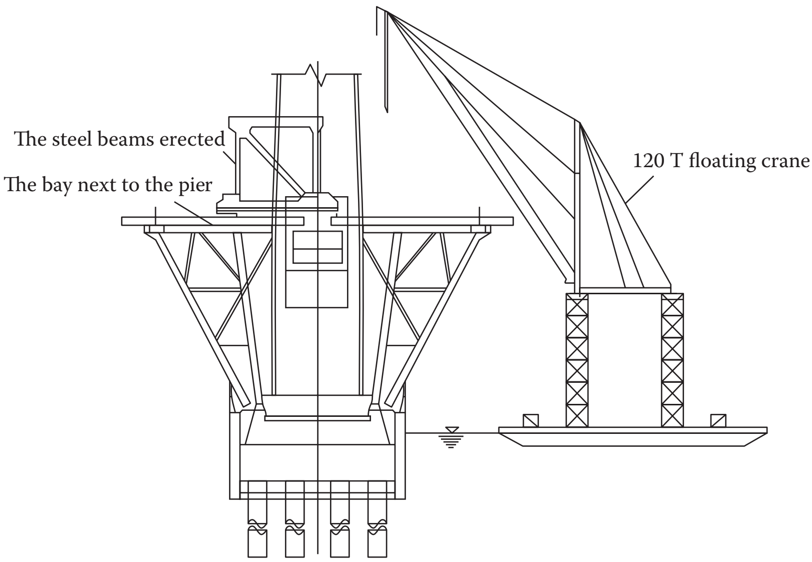

FIGURE 4.16 Interpanel steel truss segment erection over pier top. (From Hu, H.Z., *Bridge Constr.*, 3, 1–4, 2007.)

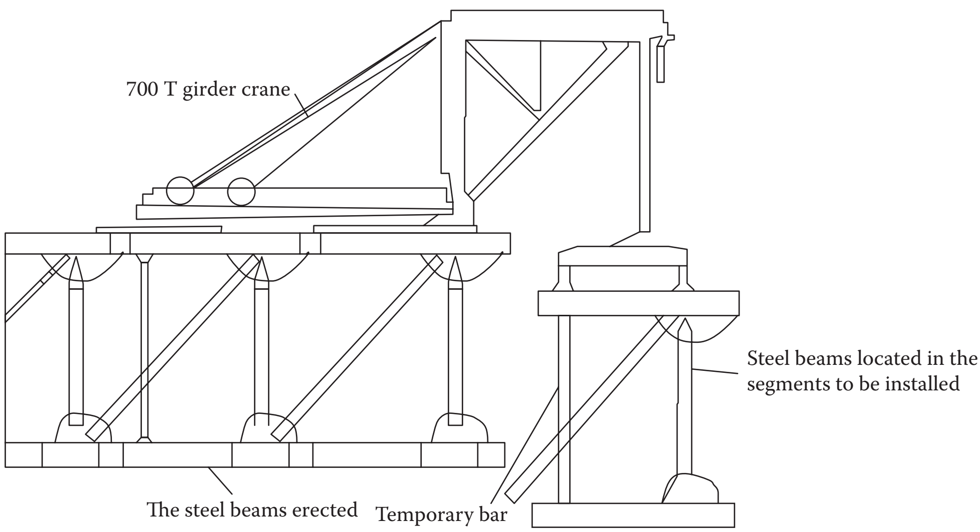

FIGURE 4.17 Whole segment lifting. (From Hu, H.Z., *Bridge Constr.*, 3, 1–4, 2007.)

4.2.4.2.2 Erecting Whole Interpanel Segment

The N-shaped whole interpanel steel truss segment and the orthotropic plates were assembled in advance, and the temporary members were installed to form a stable segment system. The most heaviest truss segment, that is, 651 ton, was erected by a 700-ton crane. The crane was equipped with adjustable angle, length, and width to erect the segment accurately as illustrated in Figure 4.17.

4.2.4.2.3 Steel Truss Girder Erection

The steel truss girders in the flood zones of the river were assembled and erected using falsework and a 75-ton gantry crane as illustrated in Figure 4.18.

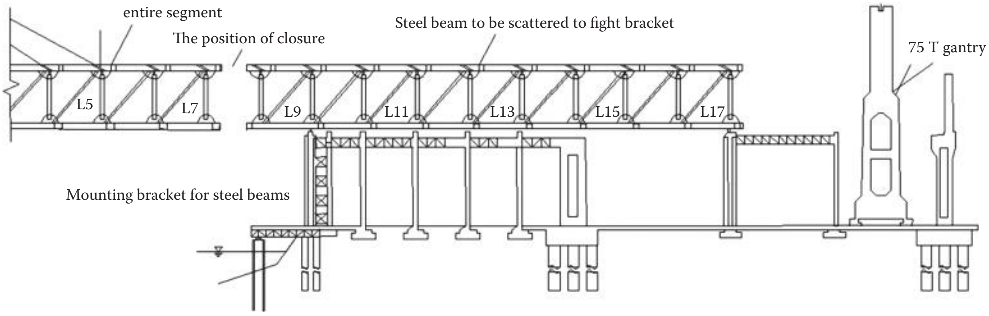

FIGURE 4.18 Steel truss girder erection on riverbank. (From Hu, H.Z., *Bridge Constr.*, 3, 1–4, 2007.)

4.3 Suspension Bridges

4.3.1 Anchorage Construction

The unique and innovative features of three large suspension bridges, Jiangyin Bridge, Yangluo Bridge, and Runyang Bridge, are presented in the following discussion.

4.3.1.1 Super Large Caisson Foundation Construction of Anchorage

Jiangyin Bridge, 1385 m main span length and 190 m tower height, is located in Huangtian Harbor, Jiangyin City, Jiangsu Province. The main tower is a portal frame structure consisting of two reinforced concrete hollow columns and three crossbeams. The superstructure is 36.9 m wide and has 3-m high steel streamline box girder carrying three highway traffic lanes in each direction.

The north anchorage is one of the four largest members in Jiangyin Bridge, carrying 64,000-ton tension force of the cable, which is resolved into a horizontal force of 55,000 ton and a vertical uplift force of 27,000 ton. The foundation of anchorage is a deep reinforced concrete caisson of 36 compartments as illustrated in Figure 4.19.

The caisson of a concrete volume 58,000 m^3 is divided into 11 segments. The first segment, 13 m high, is made of concrete with steel external shell and the rest of segments, 5 m high, are made of reinforced concrete. The bottom sealed concrete is 8 m thick and concrete volume is 22,000 m^3. The construction methods (Ouyang et al. 2004) and processes are described in the following discussion.

4.3.1.1.1 Construction Procedure

During the super large anchorage caisson construction, a procedure was developed considering the geographical location, geotechnical conditions, construction time frame, and Chinese construction specifications. The procedure includes site leveling, soft foundation treatment, manufacture and installation of the steel caisson; casting concrete within the first segment of steel shell caisson; manufacture of the second segment reinforced concrete caisson dewatering and drainage sinking for the first time; manufacture of the third segment caisson; dewatering and sinking for the second time; manufacture of segments 4, 5, and 6; drainage sinking for the third time; manufacture of segments 7, 8, 9, and 10; sinking without drainage for the fourth time; manufacture of segment 11; sinking without drainage for the fifth time; reaching the design elevation; and clearing the foundation, sealing the bottom concrete.

4.3.1.1.2 Manufacture of Caisson

Manufacture and Installation of Steel Shell Caisson To enhance the stiffness of the first segment of caisson made of concrete-filled steel shell, it was divided into 63 components and fabricated, assembled, and welded on a platform. The size of assembled steel shell caisson is 69.2 × 51.2 × 8 m and the concrete is 5730 m^3.

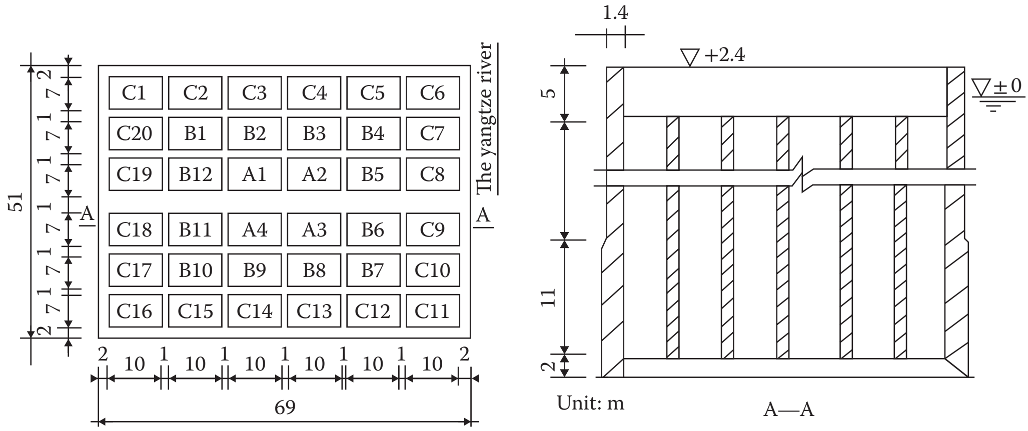

FIGURE 4.19 Caisson structure, Jiangyin Bridge.

Casting Concrete within Steel Shell Caisson Considering a big area and many compartments of the caisson, concrete was cast in 6 casting areas at 175 positions continually, horizontally layer by layer, evenly, and completed.

Manufacture of Reinforced Concrete Caisson Segments 2 ~ 11 To meet the design demand of 5-m high caisson segment, the reinforced concrete caisson segments 2 ~ 11 were extended for five times.

4.3.1.1.3 Sinking of Dewatering Construction

The plan area of the caisson is 69 × 51 m with 36 compartments. The sinking of dewatering construction means that the environment of construction remains dry and the high-pressure water jets are used to mix soils in mud that can be pumped out of the caisson. The 36 compartments were grouped into 3 zones named A, B, and C as illustrated in Figure 4.20, in which Zone A was a suspended area, Zone B was a sinking area, and Zone C was an adjustment area. Pumping soil mud under Zone A diminished the friction resistance of caisson walls so that the caisson sunk by the self-weight.

4.3.1.2 Deep Foundation Pit Construction of Anchorage

New techniques and processes for shoring and dewatering applied in deep foundation pit construction of anchorages of the Runyang Bridge and the Yangluo Bridge are presented in the following discussion.

4.3.1.2.1 New Shoring Systems

Runyang Bridge has a total length of 35.66 km carrying three highway traffic lanes in each direction and 5.3 billion RMB total investments. The main span of the Runyang Bridge is a suspension bridge with 470 + 1490 + 470 m span arrangement, and its tower is 209.9 m high, and the diameter of two cables is 0.868 m. In the deep foundation construction of north anchorage, several shoring systems including underground continuous walls, internal supports of reinforced concrete, sand piles, and high-pressure grouting technique were used.

The main spans of the Yangluo Bridge is a suspension bridge of 1280 m span, and its south anchorage, a gravity anchorage whose foundation is a deep-buried frame anchor body of circular spread foundation. The foundation pit has 73 m external diameter, 1.5 m wall thickness, and 61.5 m depth, and its shoring structure is a circular underground continuous wall of 45-m maximum excavation depth with inner lining as illustrated in Figure 4.21.

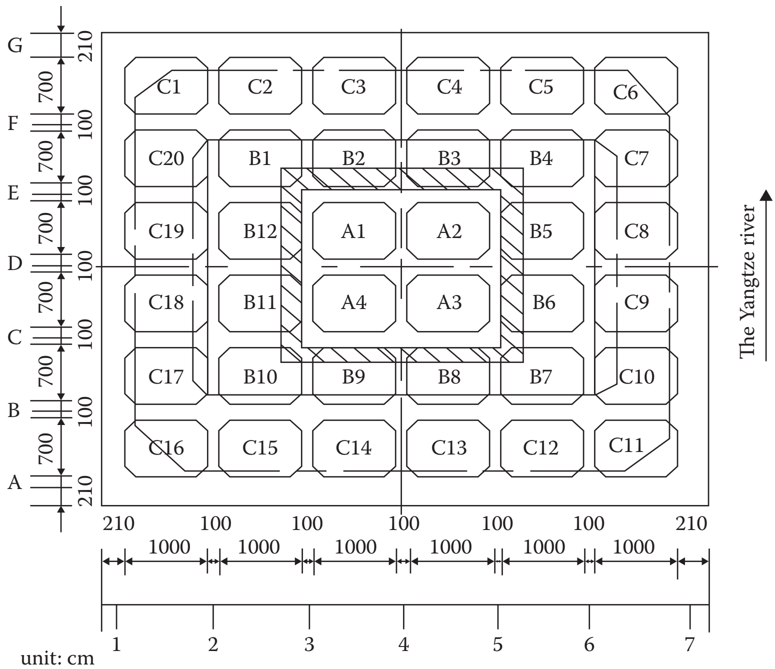

FIGURE 4.20 Partition of caisson sinking, Jiangyin Bridge. (From Hu, D.Y., *Guangxi Commun. Sci. Technol.*, 28, 2003.)

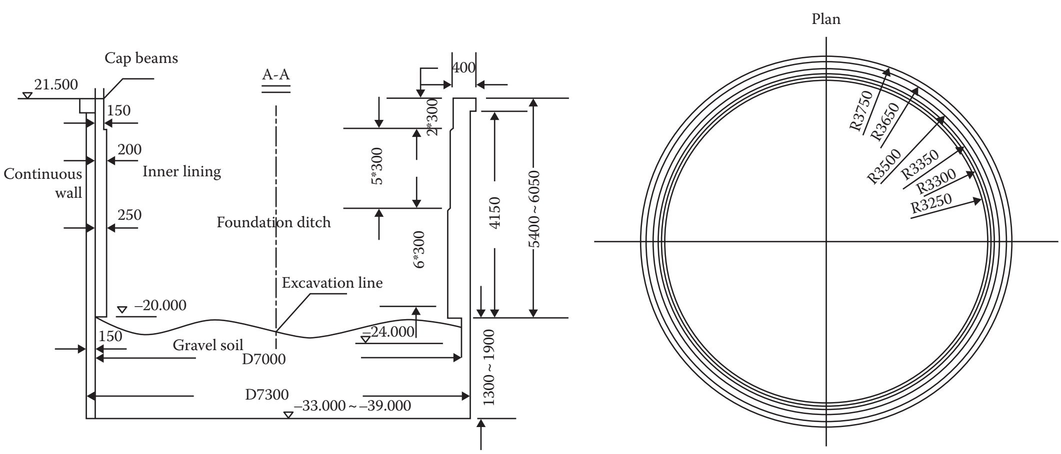

FIGURE 4.21 Shoring structure layout of south anchorage, Yangluo Bridge. (From Xu, G.P., *Highway*, 5, 2009.)

4.3.1.2.2 Sealing Water and Dewatering Systems

The purposes of lowering water level by dewatering in the foundation pit soil and the bottom of foundation pit are to (1) reduce the pore water pressure of viscous soil, (2) increase the effective stress, (3) accelerate the consolidation and drainage of soft soils, (4) improve the bearing capacity of foundation pit and the unexcavated soil resistance to the shoring structure, (5) improve equipment operation conditions, and (6) avoid the seepage failure such as sand gushing and piping during excavation. The special features of sealing water and dewatering used in anchorage foundation pits of the Yangluo Bridge and the Runyang Bridge will be discussed. Since the large foundation pit of the Yangluo Bridge south anchorage (Zhang 2006) is located in unique hydrological and geological conditions, waterproof and drainage systems became the most important factor during the foundation pit excavation as illustrated in Figure 4.22.

The geology of the north anchorage foundation pit of the Runyang Bridge is a typical double-layer structure. The upper layer is a weakly permeable cohesive soil with low strength and the lower layer is a highly permeable gravel layer with high strength. Since the underground water of the lower layer comes from the Yangtze River directly, the complex geological conditions made construction of the shoring structures and the deep foundation pit much more difficult. In the north anchorage construction, the bottom sealed scheme including the grout curtain, drainage pressure-relief holes with reliable reversed filter facilities were used for a waterhead of 52 m. Water-descending and drainage scheme consisting of precipitation wells and gravel seepage wells was applied during excavation and lining concrete construction. The water-descending and drainage scheme consists of pressure-relief drainage holes and blind drainage ditch, which was applied during the anchorage concrete construction. Since grout curtains were used to block rock faults and cracks, the seepage water was reduced effectively and the seepage paths were lengthened. The pit excavation and concrete pouring were executed in dry conditions due to a successful control of escaping hydraulic forces, uplift pressure, and osmotic pressure and to ensure the antifloating stability of anchorage concrete.

The second water sealing structure was formed 23 m above from the underground continuous wall by using double-row high-pressure jet grouting. Six deep wells within the foundation pit were drilled into the bedrock to lower the groundwater level. Double-row dewatering wells and submersible pumps were used in the outside foundation pit to make the water level in the outside pit −20 m and the water level difference within 30 m. The dewatering in the outside foundation pit reduced the lateral water pressure on the foundation pit and also improved the stress conditions of underground continuous walls.

The vacuum well-point system was applied in the south anchorage foundation pit construction of the Runyang Bridge. The plane size of the south anchorage foundation pit is 69.0 × 51.0 m; the base elevation is −26 m (Yellow Sea elevation system); the depth of excavation is 29 m. The shorting structure has 140 soldier piles with 150 cm diameter, 35 m pile length, and 6.0 m rock-embedded depth. The foundation pit was surrounded with −25 ~ −28°C circulating salt cooling water to form a 130-cm frozen soil wall in the outside of the piles to isolate water. Since the geological investigation data of the south anchorage area show that the waterhead still existed at a position of more than 20 m above the pit bottom, new vacuum deep well-point system as illustrated in Figure 4.23 was designed for the foundation pit to avoid piping and water leakage during construction.

4.3.2 Tower

Jiangyin Bridge and Yangluo Bridge are two well-known suspension bridges in China and their characters and innovations will be discussed with respect to two aspects of tower construction and crossbeam construction.

4.3.2.1 Tower Construction

The tower height of the Jiangyin Bridge is 186.926 m and the tower top elevation is 192.926 m. The tower is a portal concrete frame structure composed of two-row tower columns, tower cap, and three crossbeams. The transverse width between two columns with a slope of around 1/50, inclined inward vertically, is invariant. The section size is changed evenly from the bottom to the top along the bridge with a slope of around 1/60. The bottom column section size is 6 × 14.5 m. The top column section size is 6 × 10 m. The tower cap section size is 6 × 11 m.

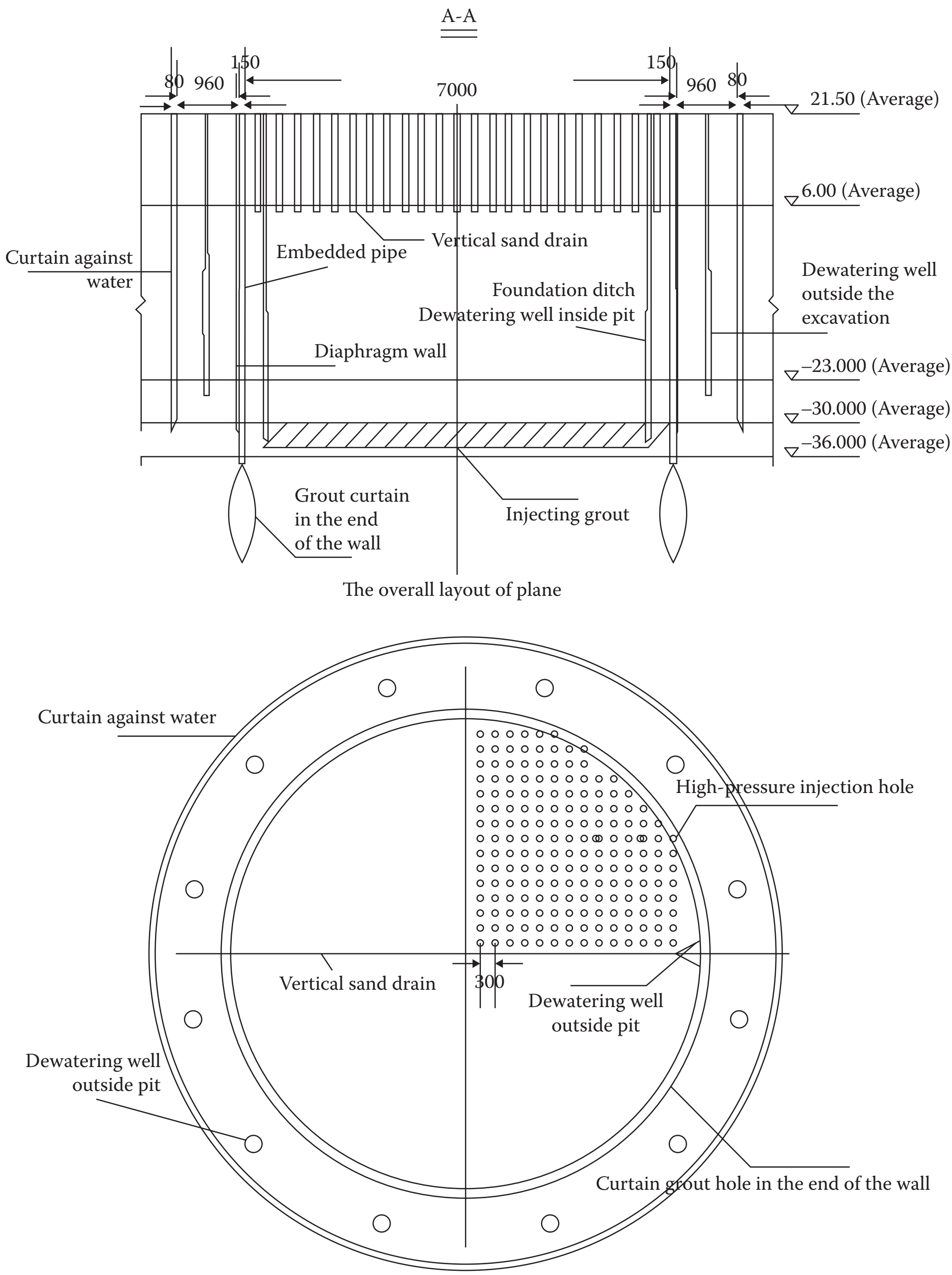

FIGURE 4.22 Sealing water, dewatering, and drainage system layout of foundation pit, Yangluo Bridge. (From Jiang, Z.Z., *Found. Hydraul. Eng.*, 6, 2006.)

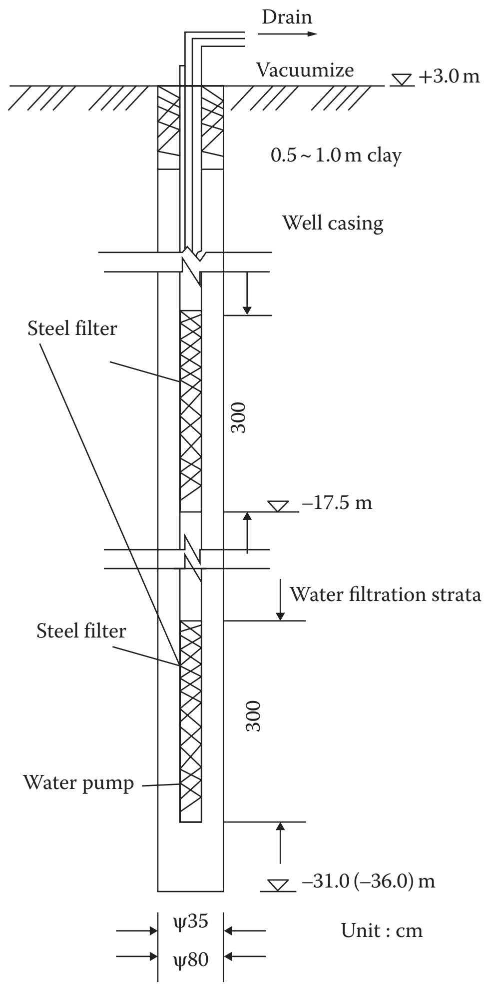

FIGURE 4.23 Structure of vacuum well-point system.

4.3.2.1.1 Construction Characters

1. A 8.5-m-high solid concrete segment in the tower bottom was built using scaffold formwork to preprevent temperature cracking of mass concrete. The rest of the hollow tower was built by climbing formwork and turnover formwork with a 5.6-m-high segment.
2. The formwork construction was required to meet the inclination of the tower column, less than 1/3000 of the tower height as illustrated in Figure 4.24.
3. The lower crossbeam was built with the tower column synchronously; the middle and upper crossbeams were built with the tower column asynchronously.
4. The infrared range finder was used to the survey of the tower axis. Other methods were also used to eliminate various measurement inaccuracies induced by wind pressures, temperature deformations, and uneven settlements.
5. The reinforcement steel were placed in their desired location using stiff skeletons.
6. Concrete mixture was designed by considering pumping, delayed hardening, early strength, and low heat performance of massive concrete.

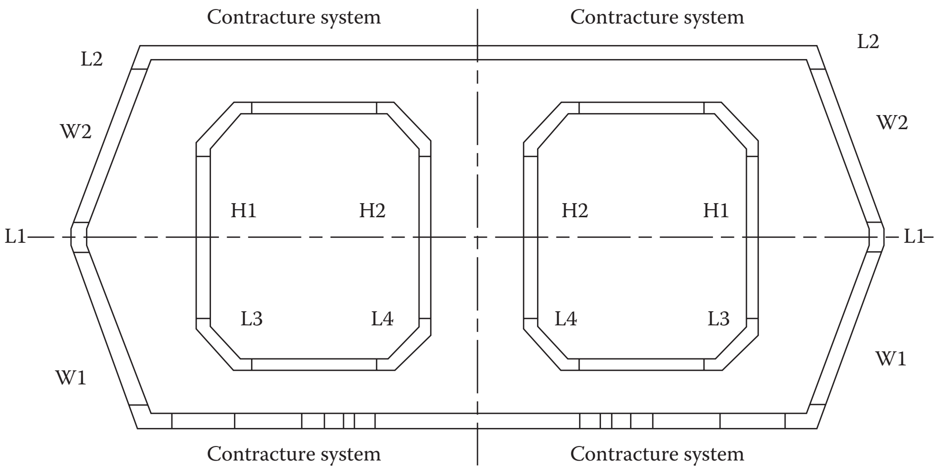

FIGURE 4.24 Plane layout of formwork, Jiangyin Bridge.

4.3.2.2 Crossbeam Construction

The crossbeam of the Yangluo Bridge tower is a hollow prestressed concrete structure. It was precast in the factory and erected at the project site. Finally, the wet joints were prestressed between the crossbeam and tower. The crossbeam structure and falsework installation sequence of the Yangluo Bridge are illustrated in Figures 4.25 and 4.26, respectively.

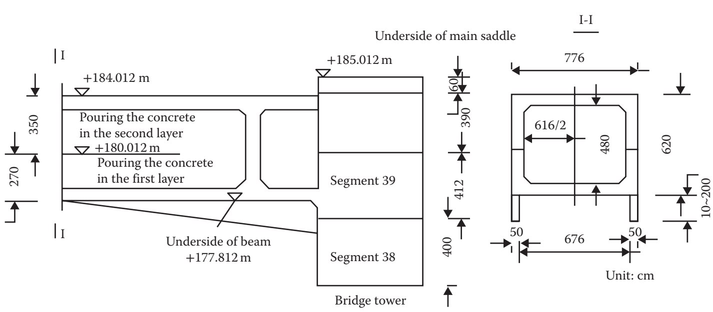

FIGURE 4.25 Crossbeam structure and layered concrete, Yangluo Bridge. (From Zhang, M. et al., *Bridge Constr.*, 6, 2006.)

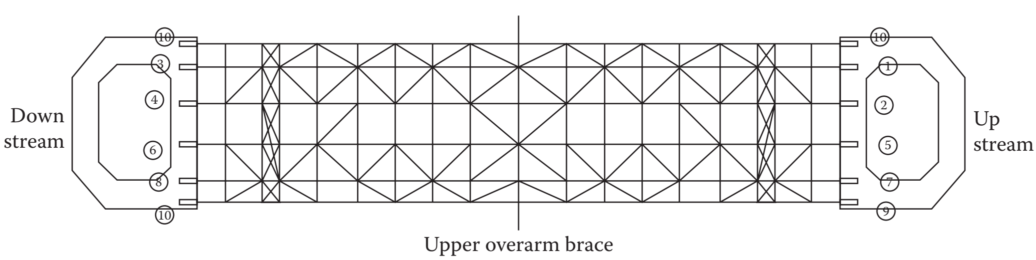

FIGURE 4.26 Falsework installation sequence of crossbeams, Yangluo Bridge. (From Zhang, M. et al., *Bridge Constr.*, 6, 2006.)

References

BSRI. 1996. *Suspension Bridge*. Scientific and Technical Documentation Press, Beijing, China (in Chinese).

CNMOR. 2005. *Interim Construction Specifications for Passenger Railway Bridges and Tunnels, Railway Construction* [2005] No.160. China Ministry of Railways, Beijing, China (in Chinese).

Gong, Z. G. 2004. "Hong Kong Stonecutters Bridge Main Span," *Bridge Design and Engineering*, (2)L: 22–27; (7): 60–61 (in Chinese).

He, F. X. and Jiang, X. X. 2002. "Floating Transport of Pier-10 Double Wall Steel Cofferdam," *Western China Exploration Engineering*, Special Edition, (001): 420–421 (in Chinese).

Hu, D. Y. 2003. "The Approaches for the Construction of Super Large Open Caisson Foundation of the North Anchorage on the Yangtze River Highway Bridge at Jiangyin," *Guangxi Communication Science and Technology*, 28(3) (in Chinese).

Hu, H. Z. 2007. "Construction Schemes for Superstructure of Main Bridge of Wuhan Tianxingzhou Yangtze River Rail-cum-Road Bridge," *Bridge Construction*, (3): 1–4 (in Chinese).

Huang, J. P. and Xu, Z. H. 2007. *Design and Construction of Stonecutters Bridge, 2007 National Bridge Conference*. Bridge and Structural Division, China Highway Association, Beijing, China (in Chinese).

Jiang, Z. Z. 2006. "Innovation of South Anchorage Continuous Wall Foundations of Yangluo Yangtze River Highway Bridge," *Foundation of Hydraulic Engineering*, 6 (in Chinese).

Li, J. P., Yao, X. Y. and Wang, S. L. 2001. "The Fabrication Technique of Steel Girders for Wuhu Yangtze River Bridge," *Bridge Construction*, (5): 52–55 (in Chinese).

Li, J. T. and D, Z. C. 2010. "Construction for Large-Diameter Bored Piles with Two-Wall Steel Cofferdam of Yangtze River Bridge," *Highway*, (10): 10–14 (in Chinese).

Li, J. T. and Qin, S. Q. 2006. "Technical Innovations of Construction of Double Wall Steel Boxed Cofferdam Foundations for Main Piers of Tianxingzhou Yangtze River Bridge," *World Bridges*, (2): 17–19 (in Chinese).

Luo, R. H. 2003. "Construction of Larger Double Wall Steel Cofferdam of Wuhu Yangtze River Bridge," *Highway*, (4): 44–46 (in Chinese).

Luo, R. H. and Yu, Y. J. 2002. "Design and Construction of Double Wall Steel Cofferdam Anchor System of Wuhu Yangtze River Bridge," *Railway Standard Design*, (9): 32–35 (in Chinese).

Ouyang, X. Y. et al. 2004. "Dewatering Design and Construction of South Anchorage Deep Foundation Pit of Runyang Bridge," *Bridge Construction*, (4): 50–53 (in Chinese).

Qin, S. Q., Gao, Z. Y. and Pan, D. F. 2007. "Research of Key Techniques for Wuhan Tianxingzhou Yangtze River Rail-cum-Road Bridge," *Bridge Construction*, (1): 1–20 (in Chinese).

Xu, G. P. 2009. "Key Techniques in Design of Wuhan Yangluo Yangtze River Bridge," *Highway*, (5) (in Chinese).

Xu, Z. H. and Huang, J. O. 2004. "Optimum Design of Stonecutters Bridge," *Proceedings of 16th National Bridge Conference*, Wuhan, China (in Chinese).

Yang, L. Q. 2007. "Construction of Deck Slabs of Wuhu Yangtze River Bridge," *World Bridges*, (1): 29–31 (in Chinese).

Ye, M. X. and Hou, W. Q. 1999. "Experimental Investigation of Composite Truss Girder of Wuhu Highway and Railway Yangtze River Bridge," *Engineering Mechanics*, Special Edition, 214–219 (in Chinese).

Yuan, R. 2008. "Construction of Pier-2 Tower Double Wall Cofferdam of Wuhan Tianxingzhou Yangtze River Bridge," *Technology Innovation Report, Wafang Data*, (25) (in Chinese).

Zhang, M., Tao, J. S. and Liu, C. Y. 2006. "Design and Construction of Prestressing Steel Bracing for Upper Cross Beam of North Tower of Wuhan Yangluo Changjiang River Highway Bridge," *Bridge Construction*, (6) (in Chinese).

Zhang, S. J. 2006. "Key Techniques for Construction of South Foundation Works of Yangluo Yangtze River Highway Bridge," *World Bridges*, (2): 20–23 (in Chinese).

Zhao, S. Y. and Li, X. T. 2001. "Manufacturing and Installation of Steel Beam of Wuhu Yangtze River Bridge," *China Railway Science*, 22(5): 107–110 (in Chinese).

Zhou, M. B. 2000a. "New Techniques in Construction of Wuhu Yangtze River Bridge," *Bridge Construction*, (2): 14–19 (in Chinese).

Zhou, M. B., Liu, Z. M. and Wang, B. M. 2003. *Suspension Bridge Handbook*. China Communications Press, Beijing, China (in Chinese).

Zhou, S. Z. 2000b. "Key Techniques in Construction of Jiangyin Yangtze River Bridge," *Bridge Construction*, (2): 20–24 (in Chinese).

Zuo, H. X. and Zhang, Z. 2009. "Installation Techniques Main Bridge Stay Cables of Wuhan Tianxingzhou Yangtze River Rail-cum-Road Bridge," *Bridge Construction*, (4): 5–8 (in Chinese).

Relevant Websites

http://online.whjs.gov.cn/node/node_545.htm
http://photo.wuhu.cc/120307/25380.htm
http://hkbus.wikia.com/wiki
http://civil.seu.edu.cn/s/321/t/1688/a5/2f/info42287.htm
http://www.cots.com.cn/JingDian/JiangYinChangJiangDaQiao.html
http://www.whjt.gov.cn/jtxh2011/list .asp?articleID=30844

5

Cable Force Adjustment and Construction Control

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5.1 Introduction

Due to the excellent performance, aesthetic appeal, and economic advantages of cable-stayed bridges, they have become one of the most competitive bridge types for long-span bridges (Tang 1994). Many cable-stayed bridges have been built around the world in the past 60 years. There are three cable-stayed bridges with main spans longer than 1000 m (3280.8 ft.). The Sutong Bridge in China with a main span of 1088 m (3569.6 ft.), completed in 2008, is the longest cable-stayed bridge in the world (Zhang and Chen 2010). The Stonecutters Bridge in Hong Kong with 1018 m (3339.9 ft.) main span opened to traffic in 2009. The Eastern Bosphorus Strait Bridge with a cable-stayed main span of 1104 m (3622.0 ft.) opened to traffic in mid-2012. With the advent of high-strength materials for use in cables and the development of digital computers for structural analysis as well as of cantilever construction method, a great progress has been made in cable-stayed bridges (Walther et al. 1996). More cable-stayed bridges with larger spans are now in planning.

Under permanent load, the cable force can be adjusted and the cable-stayed bridge can be regarded as a large span externally prestressed structure. The cable stays not only provide elastic support to the main girder but they are also employed to jack the cable to balance the external loads.

Cable-stayed bridges are featured for their ability to have their behavior adjusted by cable stay forces (Walther et al. 1996). By adjustment of cable forces, the internal force distribution can be optimized to a state where the girders and towers are compressed with small bending moments. Thus, the performance of the materials used for girders and pylons can be efficiently utilized (Kasuga et al. 1995; Chen 2003).

During construction of a cable-stayed bridge, there are two kinds of errors encountered frequently: one is the tension force error in the jacking cables and the other is the geometric error in controlling the elevation of the girder (Qin 2007). During construction, the structure must be monitored and adjusted properly, otherwise errors may accumulate, the structural performance may be substantially influenced, or safety concerns may arise. With the widespread use of innovative construction methods, construction control systems play a more and more important role in the construction of cable-stayed bridges.

During construction, there are two ways of adjustments: adjustment of the cable forces and adjustment of the girder elevations (Chen 2003; Han et al. 2003; Li et al. 2009). The cable force adjustment may change both the internal forces and the configuration of the structure, while the elevation adjustment changes only the length of the cable and does not induce any change in the internal forces of the structure.

This chapter deals with two topics: cable force adjustment and construction control. The methods for determining the designed cable forces are discussed in Section 5.2. A presentation of cable force adjustment is given in Section 5.3. A simulation method for construction process of prestressed concrete (PC) cable-stayed bridge is illustrated in Section 5.4, and a construction control system is introduced in Section 5.5. Finally, an engineering example is briefly discussed in Section 5.6.

5.2 Determination of Designed Cable Forces

For a cable-stayed bridge, the stress state in the girder subjected to dead load is determined by tension forces in the cable stays. Generally, the cable tensions are so chosen that bending moments in the girders and pylons are eliminated or at least reduced as much as possible. Thus, the girders and pylons would be mainly subjected to compression under dead loads.

During construction period, the segment of the girder corbels by cable stays and each cable placed supports approximately the weight of one segment, with a length of the longitudinal distance between two adjacent stays. In the final state, the effects of added dead loads such as wearing surface, curbs, and railings as well as traffic live loads must also be taken into account. For a PC cable-stayed bridge, long-term effects of concrete creep and shrinkage must also be considered (Walther et al. 1996).

There are several methods available for determining the designed cable forces, such as continuous girder on rigid supports (Chen 2003; Qin 2007), zero displacement at cable girder connections (Xiao and Lin 1993), stress-free state control (Qin 2007), internal force balance method, optimization method (Yan et al. 2003), and so on.

5.2.1 Continuous Beam on Rigid Support Approach

Assume that the main girder under the dead load behaves like a continuous beam and the inclined stay cables provide rigid support for the girder. Thus, vertical components of the forces in stay cables are equal to the support reactions obtained on this basis. The tensions in the anchorage cables are made so that the pylons are not subjected to large bending moments when the dead loads are applied.

This method is widely used in preliminary design of cable-stayed bridges. In this method, the moments in the girder are small and evenly distributed. This is especially favorable for PC cable-stayed bridges because the redistribution of internal forces due to the effects of concrete creep could be reduced.

5.2.2 Zero Displacement Approach

In this approach, it is assumed that vertical displacements at the connections of cable stays with the girder are equal to zero. The tension for each cable is obtained with the restrain equation of zero displacement. When a cable-stayed bridge is constructed on full false-work form, the results of cable tensions are the same as those from the approach of continuous girder on rigid support.

The preceding two approaches are suitable for cable-stayed bridges with single pylon and nearly equal span lengths. For a three-span cable-stayed bridge with two pylons, if the ratio of side/main span length is around 0.5, a rational result can be obtained. However, in cases where the ratio of side/main span length is small, it may induce unfavorable bending moments in the pylons and uneven distribution of the cable tensions, especially the bending moments at the pylon foot could be quite large.

5.2.3 Internal Force (Stress) Balance Method

The basic principle of internal force balance method is to determine appropriate or reasonable values for initial tensions of the cables, to make the ratio of the maximum stress to the allowable stress at the upper flange equal to that at the lower flange for some control sections as the structure is subjected to both the dead load and the live load. In this method, it is assumed that the initial cable tensions are unknown, while the section characteristics, such as the internal forces induced by the dead loads and the live loads, are known.

Denote W_t as the section modulus of the upper flange of a control section, W_b the section modulus of the lower flange, M_a and M_i the minimum and maximum live load moments, respectively, R_t the allowable stress of the upper flange, R_b the allowable stress of the lower flange, and M_e the dead load bending moment satisfying the internal force balance requirement.

Then the upper flange stress S_t and the lower flange stress S_b can be written as follows:

$$S_t = \left(\frac{M_a + M_e}{W_t}\right) \tag{5.1}$$

$$S_b = \left(\frac{M_i + M_e}{W_b}\right) \tag{5.2}$$

According to the balance principle, it is required that

$$\frac{S_t}{R_t} = -\frac{S_b}{R_b} \tag{5.3}$$

Substituting Equation 5.1 and 5.2 in Equation 5.3 yields

$$\frac{(M_a + M_e)}{W_t R_t} = -\frac{(M_i + M_e)}{W_b R_b} \tag{5.4}$$

Then the dead load bending moment M_e can be obtained from Equation 5.4 as follows:

$$M_e = -\frac{(KM_i + M_a)}{(1+K)} \tag{5.5}$$

$$K = \frac{W_t R_t}{W_b R_b} = \frac{Y_b}{Y_t}\frac{R_t}{R_b} \tag{5.6}$$

where Y_b is the distance from the neutral axis to the upper flange and Y_t is the distance to the lower flange.

The dead load bending moment vector $\{M_e\}$ is a target vector. It consists of two parts: one is the vector $\{M_g\}$ induced by the dead load, except the cable tension, and the other is the vector $\{M_c\}$ induced by cable initial tensions only. Therefore, the dead load moment can be written as follows:

$$\{M_e\} = [A]\{T\} + \{M_g\} \tag{5.7}$$

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where $\{T\}$ is the initial tension vector of the cable stays, and $[A]$ is the influence matrix.

From Equation 5.7, we can find that

$$\{T\}=[A]^{-1}\left(\{M_e\}-\{M_g\}\right) \tag{5.8}$$

Now the initial tension vector of cable stays $\{T\}$ is derived from the internal force balanced method. $\{T\}$ is considered as the most reasonable initial tension of the cable stays. The steps to determine vector $\{T\}$ by internal force balance method are summarized as follows:

Step 1. Total dead load moment $\{M_e\}$ is calculated by Equation 5.5.

Step 2. The initial tension vector of the cable stays $\{T\}$ is determined by Equation 5.8.

If the cross-sections are made of the same type of material and the sections are symmetric with respect to the neutral axis, then we have

$$Y_t=Y_b \quad R_t=R_b \quad K=1 \tag{5.9}$$

$$M_e=-\frac{(M_a+M_i)}{2} \tag{5.10}$$

As can be seen from Equation 5.10, if the cross-sections satisfy the preceding two conditions, then the target moment is equal to half of the algebraic sum of the maximum and minimum live load moments. It means that the expected dead load moment (target moment) can be expressed by a centerline between the upper and lower bounds of the live load moment envelope with an opposite sign. Then the combined moment envelope diagram as a sum of the dead and the live load envelopes could be zero. In this case, zero line is the basic line.

Consequently, the essence of "cable force adjustment" for a cable-stayed bridge is trying to change the position of the centerline for the internal force envelope under the applied loads. Taking into account the effects of the dead load and live load, the purpose of the method is to pull the centerline of the envelopes close to zero position. Then, it can result in a very flat internal force envelope diagram so as to make the structure a satisfactory stress state.

5.2.4 Approach of Stress-Free State Control

If the nonlinear effects due to cable sags and time-dependent effects of concrete creep and shrinkage are neglected, a cable-stayed bridge could be disassembled elastically. If the lengths and curvatures of all the structural components are kept unchanged, then the internal force condition and the shape of the structure would be the same as its initial values. Based on the preceding assumption, the relation between the erection state and completion state of a cable-stayed bridge can be established (Qin 2007). However, if the nonlinearities of the structural response under permanent and living loads are considered, iteration procedures are needed for computation of cable tensions. The iteration steps are as follows:

Step 1. Based on the design profile of the cable-stayed bridge, the stress-free lengths of each cable at completion stage are calculated and the fabrication camber of main girder at stress-free state are determined. The initial camber of the main girder are set equal to the values that the design profile of the girder minus the total deflection due to various actions such as the self-weight of the girder, initial tensions of each cable stays, the effects of prestresses, and the time-dependent effects of concrete material. At the first loop, the effects due to concrete shrinkage and creep need not be included.

Step 2. When the forward assembly computation is conducted, the lengths of cables at stress-free state are taken as a control value. The cable stays could be tensioned once or more number of times based on the load condition. At the last tension, the length of a cable is adjusted to be equal to that value of the stress-free length and the elevation of the girder is set as its initial value.

Step 3. The cable tensions are adjusted to make sure that the elevation curve of the girder is smooth at the closure of the main girder.

Step 4. Due to nonlinear effects of the structure and time-dependent effects of concrete creep/shrinkage, some differences occur at the completion stage. The differences of the cable lengths are differences between the prediction values by the preceding method and the predetermined design values. Then the pre-camber of main girder and the length of the free cable are adjusted based on the cable tension and the elevation curve of the girder at the completion stage. If the differences are not tolerated, go back to step 1 and start a new computation loop.

5.2.5 Optimization Method

In optimization method of determining the stresses of the stay cables under permanent loads, the criteria (objective functions) are chosen so that the materials used in girders and pylons are minimized. When the internal forces, mainly the bending moments, are evenly distributed and small, the quantity of material reaches a minimum value. Also, the stresses in the structure and the deflections of the girder are limited to prescribed tolerances.

In a cable-stayed bridge, the shear deformations in the girders and pylons are neglected, and the strain energy can be expressed by

$$U = \frac{1}{2}\int_0^L \frac{M^2}{2\mathrm{EI}}\,\mathrm{d}x + \frac{1}{2}\int_0^L \frac{N^2}{2\mathrm{EA}}\,\mathrm{d}x \tag{5.11}$$

where EI is the bending stiffness of the girders and pylons, and EA is the axial stiffness.

It can be given in a discrete form when the structure is simulated by a finite element model as

$$U = \sum_{i=1}^{N} \frac{L_i}{4E_i}\left(\frac{M_{il}^2 + M_{ir}^2}{I_i} + \frac{N_{il}^2 + N_{ir}^2}{A_i}\right) \tag{5.12}$$

where N is the number of the girder and pylon elements, L_i is the length of the ith element, E is the modulus of elasticity, and I_i and A_i are the moment of inertia and the section area, respectively. $M_{ir}, M_{il}, N_{ir}, N_{il}$ are the moments and the normal forces in the left and right end sections of the ith element, respectively.

Under the application of dead loads and cable forces, the bending moments and normal forces of the girders and pylons are given by

$$\{M\}=\{M_{\mathrm{D}}\}+\{M_{\mathrm{P}}\}=\{M_{\mathrm{D}}\}+[S_{\mathrm{M}}]\{P_0\} \tag{5.13a}$$

$$\{N\}=\{N_{\mathrm{D}}\}+\{N_{\mathrm{P}}\}=\{N_{\mathrm{D}}\}+[S_{\mathrm{N}}]\{P_0\} \tag{5.13b}$$

where $\{M_{\mathrm{D}}\}$ and $\{M_{\mathrm{P}}\}$ are the bending moment vectors induced by dead loads and cable forces, respectively; $[S_{\mathrm{M}}]$ is the moment influence matrix; $[S_{\mathrm{N}}]$ is the normal force influence matrix, the component S_{ij} of influence matrix represents changes of moment in the ith element induced by the jth unit cable force; $\{N_{\mathrm{D}}\}$ and $\{N_{\mathrm{P}}\}$ are the normal force vectors induced by dead loads and cable forces, respectively; and $\{P_0\}$ is the vector of cable forces under dead loads.

The corresponding displacements in the girders and pylons are given as follows:

$$\{F\}=\{F_{\mathrm{D}}\}+\{F_{\mathrm{P}}\}=\{F_{\mathrm{D}}\}+[S_{\mathrm{F}}]\{P_0\} \tag{5.14}$$

where $\{F\}$ is the displacement vector,$\{S_F\}$is the displacement influence matrix, and $\{F_D\}$ and $\{F_P\}$ are the displacement vectors induced by dead loads and cable forces, respectively.

Substitute Equation 5.13a 5.13b in Equation 5.12 and replace the variables by

$$\{\bar{M}\} = [A]\{M\}, \ \{\bar{N}\} = [B]\{N\} \tag{5.15}$$

where $[A]$ and $[B]$ are diagonal matrices.

$$[A] = \text{Diag}\left[\sqrt{L_1/4E_1I_1}, \sqrt{L_2/4E_2I_2}, \ldots, \sqrt{L_n/4E_nI_n}\right]$$

$$[B] = \text{Diag}\left[\sqrt{L_1/4E_1A_1}, \sqrt{L_2/4E_2A_2}, \ldots, \sqrt{L_n/4E_nA_n}\right]$$

Then, the strain energy of the cable-stayed bridge can be represented in matrix form as follows:

$$U = \{P_0\}^T[\bar{S}]^T[\bar{S}]\{P_0\} + 2\{\bar{P}_D\}^T[\bar{S}]\{P_0\} + \{\bar{P}_D\}^T\{\bar{P}_D\} \tag{5.16}$$

where $[\bar{S}] = (\bar{S}_M, \bar{S}_N)^T = [A,B](S_M, S_N)^T, \ \{\bar{P}_D\} = \{M_D, N_D\}^T$

Now, we want to minimize the strain energy of structures, to let

$$\frac{\partial U}{\partial P_0} = 0 \tag{5.17}$$

under the following constraint conditions:

1. The stress range in girders and pylons must satisfy the following:

$$\{\sigma\}_L \le \{\sigma\} \le \{\sigma\}_U \tag{5.18}$$

   where $\{\sigma\}$ is the maximum stress vector. $\{\sigma\}_L$ and $\{\sigma\}_U$ are vectors of the lower and upper bounds.
2. The stresses in stay cables are limited so that the stays can work normally.

$$\{\sigma\}_{LC} \le \left\{\tfrac{P_{0C}}{A_C}\right\} \le \{\sigma\}_{UC} \tag{5.19}$$

   where A_C is the area of a stay, P_{0C} is the cable force, and $\{\sigma\}_{LC}$ and $\{\sigma\}_{UC}$ represent the lower and upper bounds, respectively.
3. The displacements in the girders and pylons satisfy

$$\{|D_i|\} \le \{\Delta\} \tag{5.20}$$

The left-hand side of Equation 5.20 is the absolute value of maximum displacement vector and the right-hand side is the allowable displacement vector.

Equations 5.16 and 5.17 in conjunction with the Equations 5.18 through 5.20 are a standard quadric programming problem with constraint conditions. It can be solved by standard mathematical methods.

Since the cable forces under dead loads determined by the optimization method are equivalent to that of the cable force under which the redistribution effect in the structure due to concrete creep is minimized, the optimization method is used more widely in the design of PC cable-stayed bridges.

5.2.6 Engineering Examples

5.2.6.1 Example 1

A cable-stayed bridge with steel main girder is shown in Figure 5.1. The forces of cable stays under permanent loads can be determined by the above-mentioned methods. The cable-stayed bridge has a total length of 840 m (2755.9 ft.). The main span between towers is 480 m (1574.8 ft.) and the side anchor spans is 180 m (590.6 ft.). The side spans consist of two spans of 120 m (393.7 ft.) and 60 m (196.9 ft.) with an auxiliary pier in between. The cable forces obtained by the four approaches discussed in Sections 5.2.1 to 5.2.4 are listed in Table 5.1 and comparisons are shown in Figure 5.2. As can be seen, there are no prominent differences among the cable forces obtained by the four approaches, except that in the

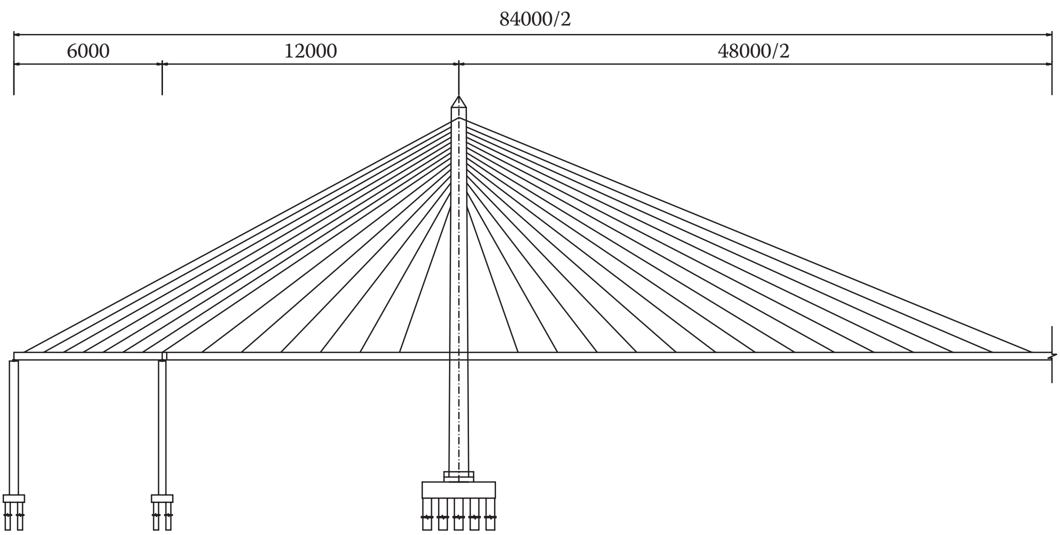


FIGURE 5.1 Elevation view of a cable-stayed bridge.

TABLE 5.1 Comparison of Cable Forces at Bridge Completion (kN)

	CBRS	IFB	SFSC	OPT		CBRS	IFB	SFSC	OPT
S14	3150	3573	3677	3480	M1	1830	1530	1351	1475
S13	3440	3482	3546	3394	M2	1700	1466	1372	1419
S12	3630	3358	3466	3348	M3	1840	1628	1657	1576
S11	3210	3318	3351	3259	M4	1760	1731	1843.5	1709
S10	2900	3097	3150	3170	M5	2050	1930	2014.5	1956
S9	2990	2922	2927.5	2834	M6	2080	2088	2198.5	2102
S8	2600	2599	2613	2519	M7	2340	2288	2359.5	2201
S7	2590	2236	2158.5	2066	M8	2440	2484	2533.5	2422
S6	2380	2190	2126.5	2082	M9	2840	2695	2626.5	2541
S5	2050	1822	2004.5	1891	M10	2780	2719	2770.5	2765
S4	1760	1683	1696	1739	M11	3020	2969	2900	2824
S3	1820	1591	1535.5	1667	M12	3050	3011	3013.5	3008
S2	1680	1399	1367	1338	M13	3260	3178	3076.5	3015
S1	1750	1326	1329	1281	M14	3380	3391	3432	3372

Notes: CBRS, continuous beam on rigid supports; IFB, inner force balanced method; M, middle span; OPT, optimization method; S, side span; SFSC, stress-free state control method.

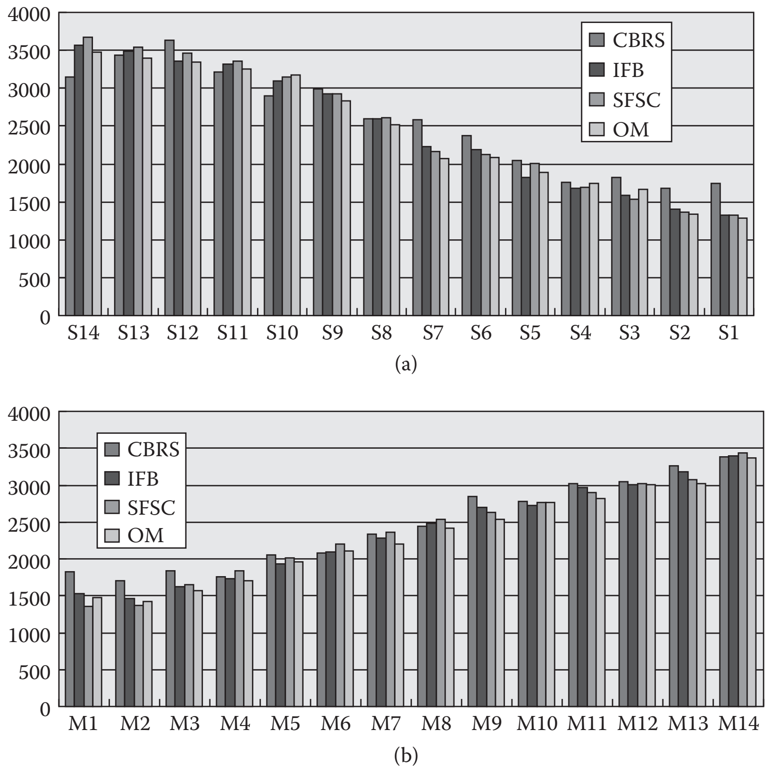


FIGURE 5.2 (a) Comparison of cable tensions in side span by various methods. (b) Comparison of cable tensions in middle span by various methods.

region near the pylon the cable forces show notable differences with various methods. For the cable stays near the supports, the cable force is evenly distributed.

Generally speaking, the differences of cable forces under dead loads obtained by the above-mentioned methods are not so significant. Continuous beam on rigid support and inner force balance methods are suitable to use for design at the completion stage. The stress-free state control method is widely used in the construction process. The optimization method based on a rigorous mathematical model is used for final design. Anyway, in practical engineering applications, the choice of the above-mentioned methods is very much dependent on the design stage and designer preference.

5.2.6.2 Example 2

The Dongsha Bridge shown in Figure 5.3 is given herein as an example (Liang 2007). Located south to the central region of Guangzhou City, China, the Dongsha Bridge is a cable-stayed bridge with single pylon. A steel–concrete girder fixed with the pylon is totally 518 m long. The spans are arranged as 338 + 72 + 56 + 52 m (1108.9 + 236.2 + 183.7 + 170.6 ft.). A part of the girder in main span, 297 m (1699.5 ft.) long, is a steel box girder. The other part, 41-m (134.5-ft.) long main span and 180-m (590.6-ft.) long side span, is a PC box girder. The girder section has triple-cell trapezoidal shape and 38 m (124.7 ft.) width and 3.3 m (10.8 ft.) height at the middle of the span. There are totally 42 pairs of cables, that is, 21 pairs for each half. The cables have 16-m (52.5 ft.) interval at the steel deck and 8-m (26.3 ft.) interval at the concrete deck.

As mentioned Section 5.2.5, for a preliminary design, the forces of the cable stays can be determined by one of the above-mentioned methods without significant differences. However, in final design, the

FIGURE 5.3 Overlook of the Dongsha Bridge, Guangzhou, China.

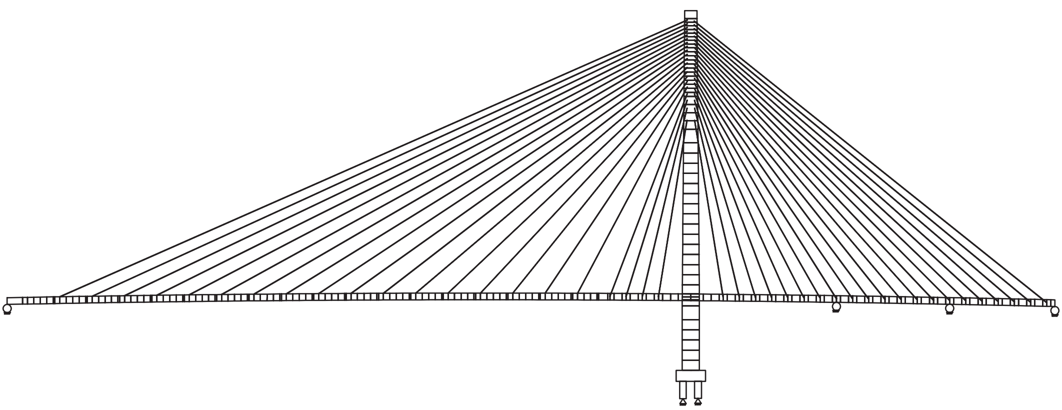


FIGURE 5.4 Finite element model.

life load and other loads such as creep and shrinkage of concrete have to be considered. Therefore, the optimization method is preferred.

In this example, the work is done for optimizing the cable forces so that the internal forces in the girders and pylons are idealized, that is, bending moments in components of the girders and pylons are made as small as possible.

A finite element model with 245 nodes and 286 elements for the bridge structure is shown in Figure 5.4. During analysis, 65 structural states in all the construction steps and a service state at completion are considered. This is a repeated design processes: to give an initial structure scheme, to compute the live load envelope, to optimize the internal force distribution, to adjust the structural scheme, and so on, until a satisfactory internal force distribution is reached. After a lot of repeated and iterative

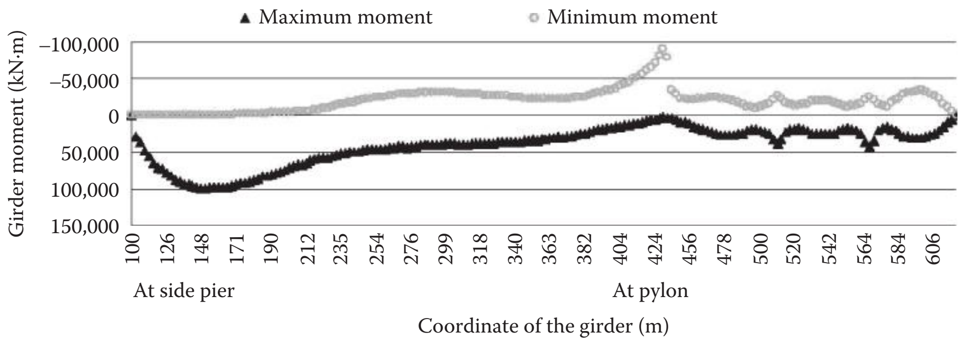


FIGURE 5.5 Bending moment envelope of the girder under life loads.

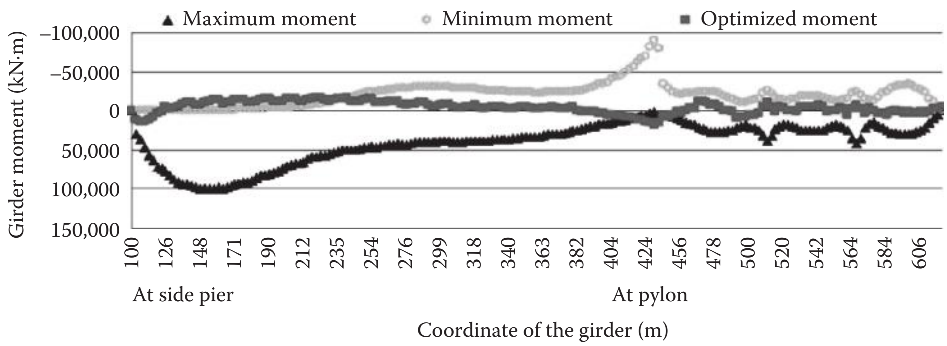


FIGURE 5.6 Optimized bending moment for the girder subject to the dead load.

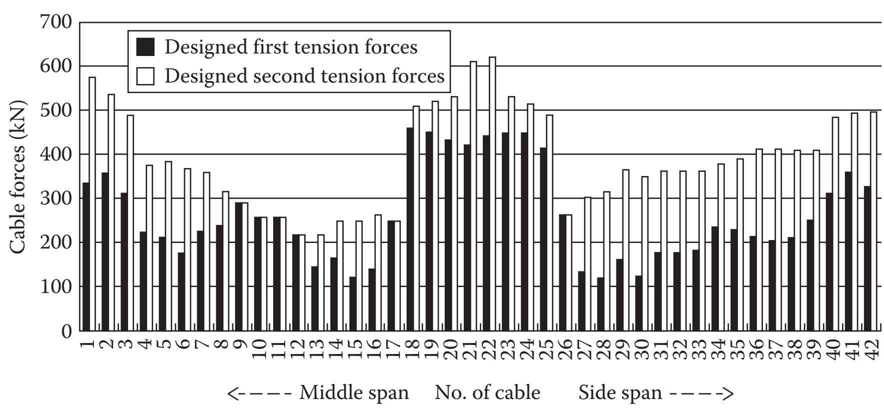


FIGURE 5.7 Designed optimized cable forces at first and second loading states.

computations, an optimized bending moment envelope for the life load is found as shown in Figure 5.5. Also, an optimized bending moment distribution for the dead load is shown in Figure 5.6. The bending moment envelope for the life load is also shown in the figure. As can be seen, the bending moments are quite small and have a reasonable distribution after optimization.

The corresponding cable forces are shown in Figure 5.7, in which the initial cable force at the first tension means that only the dead weight of the main structure is applied, while at the second tension means that all the dead loads, including the weights of the wearing, the surfacing, the curbs, the fence, and so on, are applied. For a cable-stayed bridge with steel–concrete girder if all the cables are stressed to their final design values, large internal forces may occur at the main span. It may cause unfavorable effects for the structure. Therefore, two sets of design values for the cables are given corresponding to two loading states.

5.3 Adjustment of Cable Forces

5.3.1 General

During construction, many factors may induce errors in the cable forces and elevation of the girder, such as the operational errors in tensioning stays and the errors of elevation in laying forms (Furukawa et al. 1986). Furthermore, the discrepancies of parameter values between design and reality such as the modulus of elasticity, the mass density of concrete, and the weight of girder segments may give rise to disagreements between the real structural response and the theoretical prediction (Chen et al. 1993). If the structure is not adjusted to reduce the errors during construction, the errors may accumulate and the structure may deviate away from the intended design aim. Moreover, if the errors are greater than the allowable limits, they may induce unfavorable effects to the structure. Through cable force adjustment, the construction errors can be eliminated or reduced to an allowable tolerance. In the service stage, because of concrete creep effects, cable force may also need to be adjusted; thus, an optimal structural state can be reached or recovered. In what follows, the linear programming method is introduced for cable force adjustment.

5.3.2 Linear Programming Method

Assuming that a unit amount of cable force is adjusted in one cable stay, the deformations and internal forces of the structure can be calculated by the finite element model. The vectors of change in deformations and internal forces are defined as influence vectors. In this way, the influence matrices can be formed for all the stay cables.

Assume that there are n cable stays whose cable forces are to be adjusted, and T_i ($i = 1, 2,\ldots, n$) is the adjusted value of the ith cable. Then a cable force adjustment vector is expressed as

$$\{T\} = \{T_1, T_2, \ldots, T_n\}^T \tag{5.21}$$

Denote internal force influence vector $\{P_l\}$ as

$$\{P_l\} = (P_{l1}, P_{l2}, \ldots, P_{ln})^T \qquad (l = 1,2,\ldots,n) \tag{5.22}$$

where m is the number of sections of interest, and P_{lj} is the internal force increment at section l due to a unit tension of the jth cable. Denote displacement influence vector $\{D_i\}$ as

$$\{D_i\} = (D_{i1}, D_{i2}, \ldots, D_{in})^T \qquad (i = 1,2,\ldots,k) \tag{5.23}$$

where k is the number of sections of interest, and D_{ij} is the displacement increment at section i due to a unit tension of the jth cable. Thus, the influence matrices of internal forces and displacements are given by

$$\{P\} = (P_1, P_2, \ldots, P_m)^T \tag{5.24a}$$

$$\{D\} = (D_1, D_2, \ldots, D_k)^T \tag{5.24b}$$

respectively. With the application of cable force adjustment $\{T\}$, the increment of the internal forces and displacements can be obtained as

$$\{\Delta P\} = \{P\}^T \{T\} \tag{5.25a}$$

$$\{\Delta D\} = \{D\}^T \{T\} \tag{5.25b}$$

respectively.

Denote deflection error vector $\{H\}$ as

$$\{H\} = (h_1, h_2, \ldots, h_m)^T \tag{5.26}$$

Denote internal force vector $\{N\}$ as

$$\{N\} = (N_1, N_2, \ldots, N_m)^T \tag{5.27}$$

After cable force adjustments, the absolute values of the deflection errors are expressed by

$$|\lambda_k| = \left| \sum_{i=1}^{n} D_{ik} T_i - h_k \right| \tag{5.28}$$

The absolute values of internal force errors are expressed by

$$|q_l| = \left| \sum_{i=1}^{n} P_{il} T_i - N_l \right| \tag{5.29}$$

The objective function for cable force adjustments may be defined as the errors of girder elevation, that is,

$$\min|\lambda_k| \tag{5.30}$$

The constraint conditions may include limitations of the internal force errors, the upper and lower bounds of the cable forces, and the maximum stresses in the girders and pylons. Then the optimum values of cable force adjustment can be determined by a linear programming model. The component value of the cable adjustment vector $\{T\}$ could be positive for increasing or negative for decreasing the cable forces. Introduce two auxiliary variables T_{1i}, T_{2i} as follows:

$$T_i = T_{1i} - T_{2i} \quad T_{1i} \geq 0, \ T_{2i} \geq 0 \tag{5.31}$$

Substitute Equation 5.28 in Equation 5.30, then a linear program model is established as follows:

$$\min: \lambda_k \tag{5.32}$$

$$\text{subject to: } \sum_{i=1}^{n} p_{li}(T_{1i} - T_{2i}) \geq N_l - \xi \overline{p}_l \quad (l = 1, 2, \ldots, m) \tag{5.33a}$$

$$\sum_{i=1}^{n} p_{li}(T_{1i} - T_{2i}) \leq N_l + \xi \overline{p}_l \quad (l = 1, 2, \ldots, m) \tag{5.33b}$$

$$\sum_{i=1}^{n} D_{ji}(T_{1i} - T_{2i}) \geq h_j - d_j \quad (j = 1, 2, \ldots, k) \tag{5.33c}$$

$$\sum_{i=1}^{n} D_{ji}(T_{1i} - T_{2i}) \geq h_j - d_j \quad (j = 1, 2, \ldots, k) \tag{5.33d}$$

$$T_{1i} - T_{2i} \leq \eta \overline{T}_i, \ T_{1i} - T_{2i} \geq -\eta \overline{T}_i \quad (I = 1, 2, \ldots, n) \tag{5.33e}$$

where $\overline{p}_l$ is the design value of internal force at section l, ξ is the allowable tolerance in percentage of the internal force, $\overline{T}_i$ is the design value of the ith cable force, and η is the allowable tolerance in percentage of the cable forces.

Equation 5.32 and 5.33 form a standard linear programming problem, which can be solved by mathematical software.

5.3.3 Order of Cable Adjustment

Cable force adjustment values can be determined by the preceding discussed method; however, the adjustments must be applied at the same time to all cables, and a great deal of jacks and workers are needed (Wang et al. 1996). In performing the adjustment, it is preferred that the cable stays are tensioned one by one.

When adjusting the cable force one by one, the influence of the other cable force must be considered. And for the purpose that any cable must be adjusted only one time, the adjustment values of cable force can be calculated through the influence matrix of cable force $\{\overline{T}\}$.

$$\{\overline{T}\} = [S]\{T\} \tag{5.34}$$

where $\{\overline{T}\} = \{\overline{T}_1, \overline{T}_2, \ldots, \overline{T}_n\}$ is the vector of actual adjustment value of cable tension; $[S]$ is the influence matrix of cable tension, whose component S_{ij} represents tension change of the jth cable when the ith cable changes a unit amount of force. The influence matrix $[S]$ can be computed by finite element method.

5.4 Simulation of Construction Process

5.4.1 General

Segmental construction techniques have been widely used in construction of cable-stayed bridges. In this technique, the pylon is built first. Then the girder segments are erected one by one and supported by the inclined cables hung from the pylon(s). It is evident that the profile of the main girder and the final tension forces in the cables are strongly related to the erection method and the construction scheme. It is therefore important that the designer should be aware of the construction process and the necessity to look into the structural performance at every stage of construction (Chen 2003).

To reach the design aim, an effective and efficient simulation of the construction process step by step is very necessary. The objectives of the simulation analysis are as follows:

1. To determine the forces required in cable stays at each construction stage
2. To set the elevation of the girder segment
3. To find the consequent deformation of the structure at each construction stage
4. To check the stresses in the girder and pylon sections

The simulation methods are introduced and discussed in detail in the following sections. In Section 5.4.2, the technique of forward analysis is presented to simulate the assembly process, and a flow chart for the forward assemblage method is shown in Figure 5.8. Creep effects can be considered; however, the design aim may not be successfully achieved by such a simulation because it is not so easy to determine appropriate lengths of the cable stays, while the cable lengths play a decisive role to make the final elevation to achieve the design profile. Another technique presented in Section 5.4.3 is the backward disassembly analysis, which starts with the final aim of the structural state and disassembles segment by segment in a reverse way. The disadvantage of this method is that the creep and shrinkage effects may not be able to be defined. However, values obtained from the assembly process may be used in this analysis. These two methods may be alternatively applied until convergence is reached.

It is noted that the simulation is only limited to that of the erection of the superstructure.

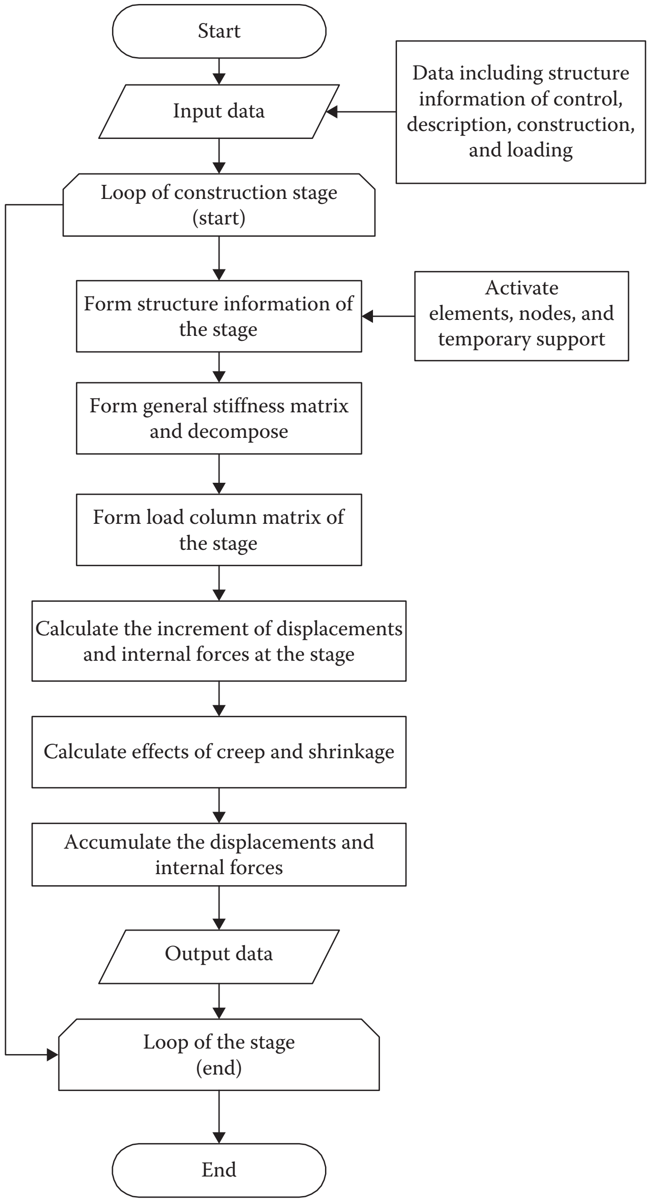


FIGURE 5.8 A flowchart of forward assembling analysis.

5.4.2 Forward Assembling Analysis

Following the known erection procedure, a simulation analysis can be carried out by finite element method (see Figure 5.8). This is the so-called forward assembling analysis. It has been used to simulate the erection process for PC cable-stayed bridges built by the cantilever method.

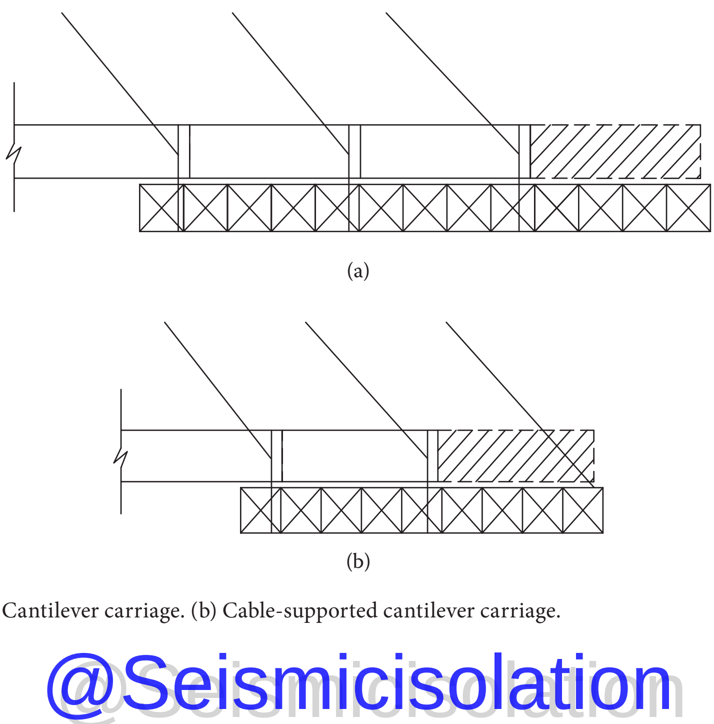
Concerning finite element modeling, the structure may be treated as a plane frame or a space frame. A plane frame model may be good enough for construction simulation because the transverse loads, such as wind action, can be generally ignored (Walther et al. 1996). In a plane frame model, the pylons and girders are modeled by some beam elements, while the stays are modeled as two-node bar elements with Ernst modules (Walther et al.1996) by which the effects of cable sag can be taken into account. The structural configuration is changed stage by stage. Typically, in one assembling stage, a girder segment treated as one or several beam elements is connected to the existing structure, while its weight is treated as a load to apply to the element. Also, the cable force is applied. Then an analysis is performed and the structure is changed to a new configuration.

In finite element modeling, several factors such as the construction loads (weight of equipment and traveling carriage) as well as the effects of concrete creep/shrinkage must be considered in detail.

Traveling carriage is specially designed for construction of a particular bridge project. Generally, there are two types of carriages corresponding to two types of erection method. One is the cantilever type as shown in Figure 5.9a. In this case, the traveling carriages are mounted at the ends of the girders, just like a cantilever, to support the next girder segment. Thus, the weight of the carriage is treated as an external load applied to the end of the girder.

With the development of multiple cable systems, the girder with lower height becomes more flexible. The girder itself is not able to carry the cantilever weights of carriage and the segment. Then an innovative erection technique is proposed. And another type of carriage is developed. This new idea is to use permanent stays to support the form traveler (Figure 5.9b) so that the concrete can be poured in situ. This method enjoys considerable success at present because of the undeniable economic advantages. Its effectiveness has been demonstrated by many bridge practices. For the erecting method using the latter type of carriage, the carriage works as a part of the whole structure when the segmental girder is poured in situ. Thus, the form traveler must be included in the finite element model to simulate construction.

With the forward assembling analysis, the construction data can be worked out. And the actual permanent state of cable-stayed bridges can be reached. Furthermore, if the erection scheme is modified during the construction period or in the case that significant construction error occurs, then the structural parameters or the temporary erection loads could be different from the values used in the design. In those cases, it is possible to predict the cable forces and the sequential deformations at each stage by utilizing the forward assembling analysis.



(a)

(b)

FIGURE 5.9 (a) Cantilever carriage. (b) Cable-supported cantilever carriage.

5.4.3 Backward Disassembling Analysis

Following a reverse way to simulate the disassembling process stage by stage, a backward analysis can be carried out also by finite element method (Chen et al. 1993; Xiao and Lin 1993) (see Figure 5.10). Not only the elevations of deck but also the length of cable stays and the initial cable tension at each erection step can be simulated by this method, and the completed state of structure at each stage can be evaluated.

The backward disassembling analysis starts with a very ideal structural state in which it is assumed that all the creep and shrinkage deformation of concrete be completed, that is, a state of

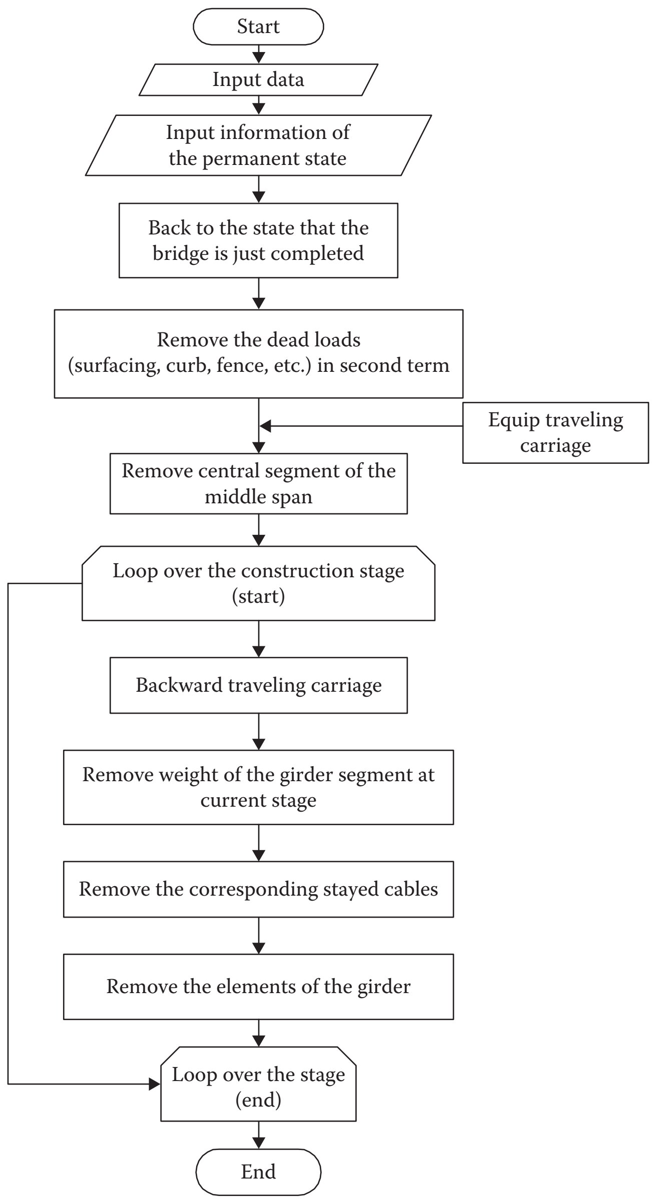


FIGURE 5.10 A flowchart of backward disassembling analysis.

5 years or 1500 days after the completion of the bridge construction. The structural deformations and internal forces at each stage are considered as ideal reference states for construction of the bridge. The backward analysis procedure for a PC cable-stayed bridge may be illustrated by the following steps:

1. Compute the permanent state of the structure.
2. Remove the effects of the creep and shrinkage of concrete that is 1500 days or 5 years old.
3. Remove the second part of the dead loads, that is, the weights of wearing, surfacing, curbs, fence, and so on.
4. Apply the traveler and other temporary loads and supports.
5. Remove the center segment so as to analyze the semistructure separately.
6. Move the form traveler backward.
7. Remove a pair of segments.
8. Remove the cable stays.
9. Remove the corresponding elements.

Repeat steps 6–9 until all the girder segments are disassembled. A flow chart for backward disassembling analysis is shown in Figure 5.10.

For the erection method using conventional form traveler cast-in-place or precast concrete segments, the crane or the form traveler may be modeled as external loads. Thus, the movement of carriage is equivalent to change in the loading position. However, for erection method using cable-supported traveling carriage, the cable stays first work as support of carriage and later, after curing is finished, are connected with the girder permanently. In backward disassembling analysis, the movement of form traveler must be related to change in the structural configuration.

The backward analysis procedure can establish the necessary data for erection at each stage such as the elevations of the girder, the cable forces, deformations of the structure, and stresses at critical sections of the girders and pylon.

One of the disadvantages of backward analysis is that creep effects cannot be estimated; therefore, forward and backward simulations should be used alternately to determine the initial tension and the length of stay cables.

5.5 Construction Control

5.5.1 Objectives and Control Means

Obviously, the objective of construction control is to build a bridge that achieves the designed aim with acceptable tolerance. During construction of a cable-stayed bridge, some discrepancies may occur between the actual state and the state of design expectation (Furukawa et al. 1986). The discrepancies may arise from elevation errors in laying forms; errors in stressing cable stays by jacks; errors of the first part of the dead loads, that is, the weights of the girder segments, and the second part of the dead loads, that is, the self-weights of surfacing, curbs, and fencing; and so on. On the other hand, a system error may occur in measuring the deflection of the girder and pylons. However, it is impossible to eliminate all the errors. There are two basic requirements for the completed structure: (1) the geometric profile should match the designed shape well and (2) the internal forces should be within the designed envelope values, specifically, the bending moments of the girder and pylons should be small and evenly distributed.

Since the internal forces of the girders and pylons are closely related to the cable forces, the basic method of construction control is to adjust the girder elevation and cable forces. If the error of the girder elevation deviated from the design value is small, it can be reduced or eliminated by adjusting the elevation of the segment without inducing an angle between two adjacent segments. In this way, we only change the geometric position of the girder without changing the internal force of the structure. When the errors are not small, it is necessary to adjust the cable forces. In this case, both the geometric position change and the internal force changes occur in the structure.

Nevertheless, to reduce the errors, cable force adjustments are not preferable because it may require a lot of time and money for adjustments. The general exercise at each stage is to find out the correct length of the cables and set the elevation of the segment appropriately. Cable tensioning is performed for the new stays only. Generally, a comprehensive adjustment of all the cables is only applied before connecting the two cantilever ends. In case a group of cables needs to be adjusted, careful planning for the adjustment based on a detailed analysis is absolutely necessary.

5.5.2 Construction Control System

To guarantee structural safety and to achieve the design aim, a monitoring and controlling system is required (Fujisawa and Tomo 1985; Chen et al. 1993; Manbe et al. 1999). A typical construction controlling system consists of four subsystems: measuring subsystem, parameter sensitivity and identification subsystem, control/adjustment subsystem, and new design value computation subsystem. Typical construction control system for a PC cable-stayed bridge (Hidemi et al. 1995; Han et al. 2003; Li et al. 2009) is briefly introduced as follows:

1. Monitoring (measuring) subsystem
   The measuring items mainly include the elevation/deflection of the girder, cable forces, horizontal displacement of the pylon(s), stresses of sections in the girder and pylon(s), modulus of elasticity and mass density of concrete, creep and shrinkage of concrete, and temperature/temperature gradient across the structure.
2. Parameter sensitivity and identification subsystem
   In this subsystem, the temperature effects are first determined and removed. Then, the sensitivity of structural parameters such as the elasticity modulus of concrete and self-weight and stiffness of the girder segment or pylon(s) is analyzed. Identifications for some sensitive parameters are performed. Through the analysis, the causes of the errors can be determined so that corresponding adjustment steps can be applied.
3. Control/adjustment subsystem
   Compare the measuring values with those of the design expectation, if the differences are lower than the prescribed limits, then go to the next stage. If it is essential to make the girder profile smooth, the elevation of the next segment could be slightly adjusted without influencing the internal forces. Otherwise, cable adjustments may be needed. The magnitude of cable tension adjustment can be determined by a linear programming model.
4. New design value prediction subsystem
   Since the structural parameters for the completed part have deviated from the designed values, the design expectation must be updated with the changed state of the structure. A simulation analysis is performed to determine new design data for the sequential construction. And the upcoming construction follows new design values so that the final state of structure can be achieved optimally.

5.6 An Engineering Example

5.6.1 General

The Panyu Bridge across the Pearl River is a PC cable-stayed bridge as shown in Figure 5.11. The bridge has been performing very well since it opened to traffic in September 1998.

Construction control of this bridge (Yan and Han 2001) is briefly discussed in this section. The elevation view of the cable-stayed bridge is shown in Figure 5.12, while the side view of the same

FIGURE 5.11 Overlook of the Panyu cable-stayed bridge, Guangzhou, China.

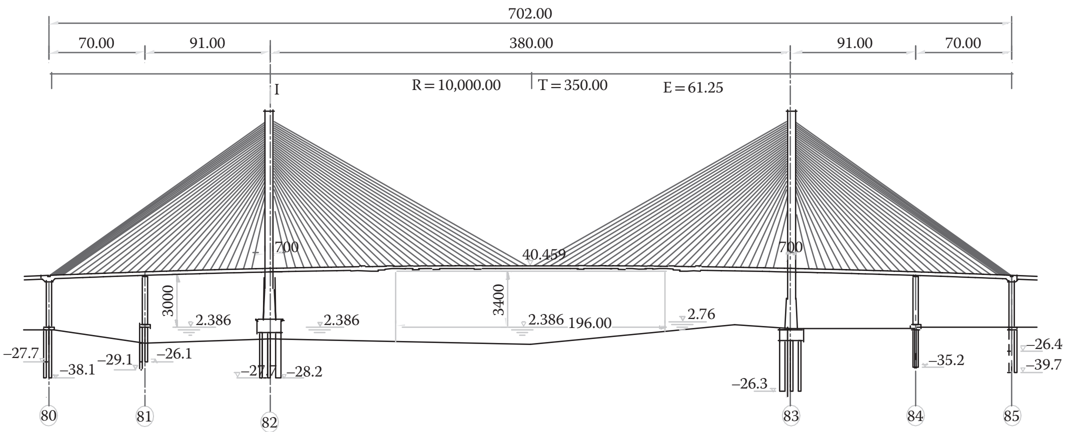


FIGURE 5.12 Elevation view of the Panyu cable-stayed bridge.

bridge is shown in Figure 5.13. The bridge girder has a total length of 702 m (2303.2 ft.). The main span between two towers is 380 m (1246.7 ft.) and the side anchor span is 161 m (528.2 ft.). The side anchor spans consist of two spans of 91 m (298.6 ft.) and 70 m (229.7 ft.) with an auxiliary pier in between. The main girder is composed of two edge girders and a deck plate. The edge girders are laterally stiffened by a T-shaped PC girder with 6 m (19.7 ft.) spacing. The edge girder is a solid section whose height is 2.2 m (7.2 ft.) and width varies from 2.6 m (8.5 ft.) at the intersection of the girder and pylon to 2.0 m (6.6 ft.) at the middle span. The deck plate is 28 cm (11 in.) thick. The width of the deck is 37.80 m (124.0 ft.) out to out with 6 traffic lanes. Spatial 112 stay cables are arranged in a semi fan configuration. All the cable stays are anchored in the mast part of the pylon. The stay cables are attached to the edge girders at 6 m (19.7 ft.) spacing. At the side anchor spans, auxiliary piers are arranged to increase the stiffness of the bridge, and an anchorage segment of deck is set up to balance the lifting forces from anchorage cables.

5.6.2 Construction Process

The bridge deck structure is erected by the balance cantilevering method utilizing cable-supported form travelers. The construction process is briefly described as follows:

Build two towers and auxiliary piers.
Work on girder erection separately.

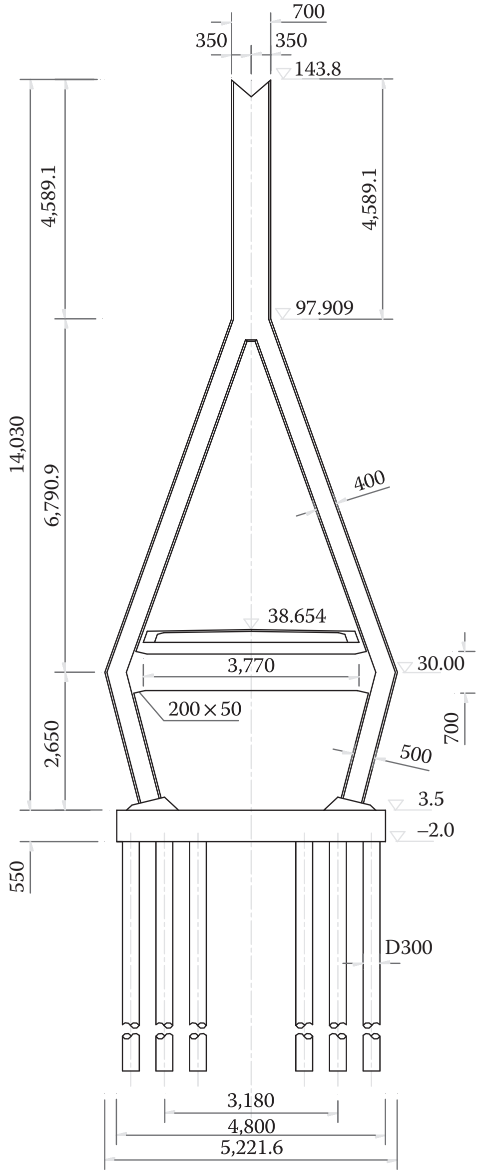


FIGURE 5.13 Site view of the tower.

Cast the first segment in situ on a timbering support.
Erect the No. 1 cable and stress to its final length.
Hoist the traveling carriages and positioning.
Erect the girder segments one by one on the two sides of the tower.
Connect the cantilever ends of the side span with the anchorage parts.
Continue erecting the girder segments in center span until two ends almost meet.
Connect the cantilever ends of the center span.
Remove the traveling carriages and temporary supports.
Connect the side spans with the auxiliary piers.
Cast pavement and set up fence, and so on.

A typical erection stage for one segment is described as follows:

Move the traveling carriage forward and set up the form at proper levels.
Erect the cables and connect with the traveler and then partially stress them.
Place reinforcement, posttensioning bars and couple the stressed bars with those of the previously completed deck segment.
Cast in situ the deck concrete.
Stress the stay cables again to adjust the girder segments to proper levels.
Stress the longitudinal and transverse bars and strands.
Loosen the connection between the stay cable and traveling carriage.
Stress the cable stays to final lengths.

The preceding erection steps are repeated until the bridge is closed at the middle span.

5.6.3 Construction Simulation

The preceding construction procedures can be simulated stage by stage as illustrated in Section 5.4.2. Since creep and shrinkage occur and the added dead weight will be loaded on the bridge girder after completion of the structure, a downward displacement is induced. Therefore, as the erection is just finished, the elevation of the girder profile should be set higher than that of the design profile and the towers should be leaning toward the side spans. In this example, the maximum value that is set higher than the designed profile in the middle of the bridge is about 35.0 cm (13.8 in.), while the displacement of pylon top leaning to anchorage span is about 9.0 cm (3.5 in.). The initial cable forces are listed in Table 5.2 to show the effects of the creep. As can be seen, considering the long-term effects of concrete creep, the initial cable forces are a little greater than those without including the time-dependent effects.

TABLE 5.2 Predicted Initial Cable Forces (kN)

No.	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
NCS	7,380	3,733	4,928	5,130	5,157	5,373	5,560	5,774	5,969	6,125
CS	7,745	3,854	5,101	5,331	5,348	5,549	5,694	5,873	6,050	6,185
No.	S11	S12	S13	S14	S15	S16	S17	S18	S19	S20
NCS	6,320	6,870	7,193	7,283	7,326	7,519	7,317	7,478	7,766	8,035
CS	6,367	6,908	7,209	7,331	7,467	7,639	7,429	7,580	7,856	8,114
No.	S21	S22	S23	S24	S25	S26	S27	S28	S29	S30
NCS	8,366	8,169	7,693	8,207	9,579	9,649	9,278	9,197	9,401	12,820
CS	8,442	8,246	7,796	8,354	9,778	9,791	9,509	9,296	9,602	12,930
No.	M1	M2	M3	M4	M5	M6	M7	M8	M9	M10
NCS	7,334	3,662	4,782	4,970	5,035	5,426	5,479	5,628	5,778	6,108
CS	7,097	3,433	4,591	4,877	5,006	5,477	5,567	5,736	5,904	6,231
No.	M11	M12	M13	M14	M15	M16	M17	M18	M19	M20
NCS	6,139	6,572	6,627	6,918	6,973	7,353	7,540	7,742	7,882	7,978
CS	6,263	6,706	6,756	7,031	7,088	7,422	7,624	7,813	7,942	8,060
No.	M,21	M22	M23	M24	M25	M26	M27	M28	M29	M30
NCS	8,384	8,479	8,617	8,833	9,175	9,359	9,394	9,480	9,641	13,440
CS	8,452	8,561	8,694	8,931	9,212	9,413	9,459	9,570	9,716	13,570

Notes: CS, with the effects of creep and shrinkage of concrete; M, middle span; NCS, without the effects of creep and shrinkage of concrete; No., cable number; S, side span.

5.6.4 Construction Monitoring and Control System

In construction practice, a construction control system is employed so as to control the cable forces and the elevation of the girder (see Figure 5.14). Before concrete casting, the reactions of the cable-supported form traveler are measured by strain gauge equipment. Thus, the weights of the four travelers used in this bridge are known.

In any case, structural safety is the most important issue. Since the stresses in the girders and pylons are related to the cable tensions, the cable forces are of great concern. Furthermore, during construction, the geometric profile of the girder is also very important. It is clear that if the elevation curve of the

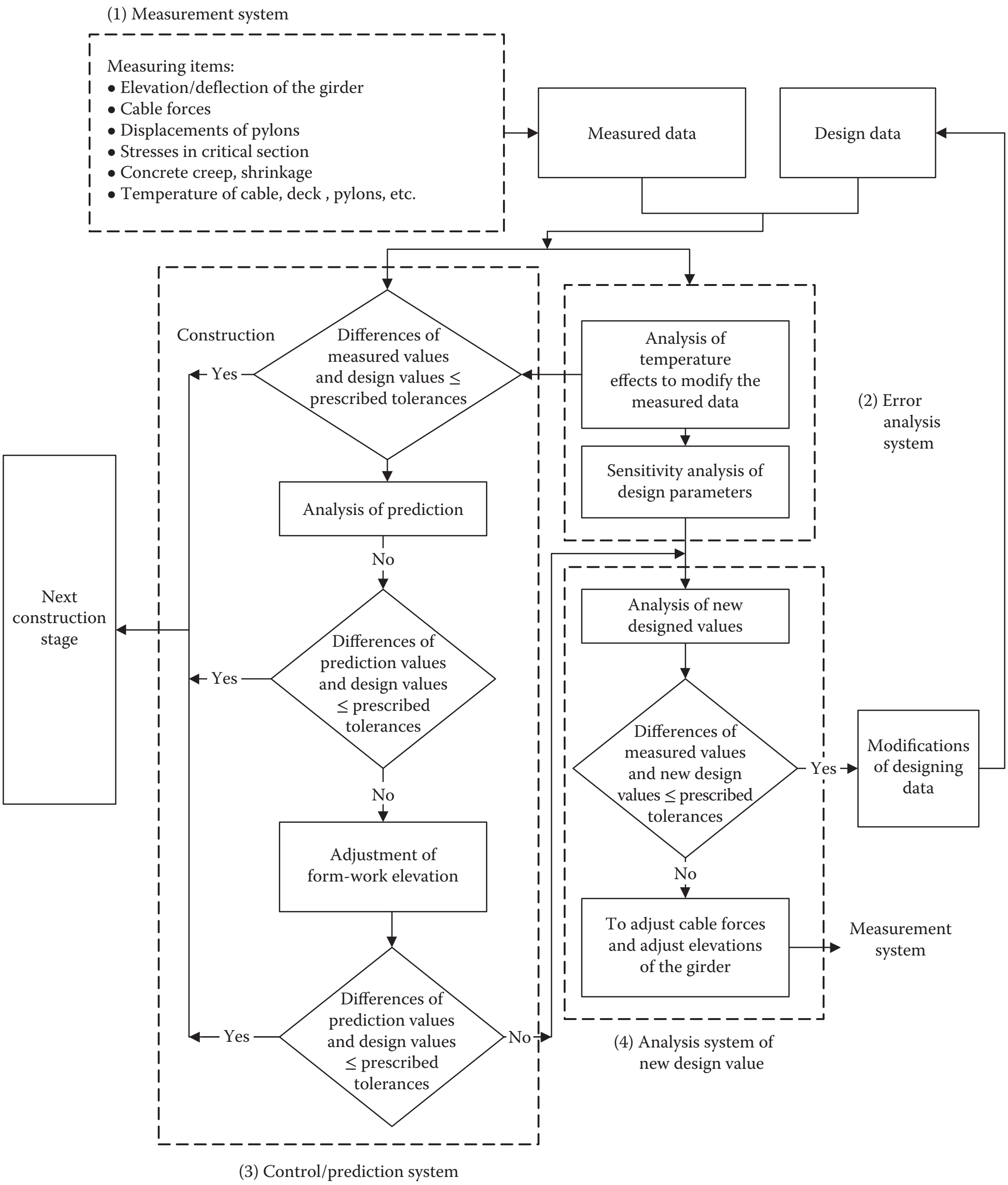


FIGURE 5.14 A typical construction system for a prestressed concrete cabled-stayed bridge.

FIGURE 5.15 Two cantilever ends are meeting very well.

girder was not smooth or the cantilever ends could not meet together, then the construction might experience some trouble. The profile of the girder or the elevation of the bridge segments is mainly controlled by the cable lengths. Therefore, the cable length must be appropriately set at the erection of each segment. It is also noticed that in the construction process, the internal forces of the structure and the elevation of the girder could vary because usually the bridge segments are built one by one, the erection equipment are placed at different positions during construction, and that some errors such as the weight of the segment and the tension force of the cable may occur. Thus, monitoring and adjustment are absolutely needed at each construction step.

A detailed simulation analysis of construction is carried out by a computer program developed by the research group. The configuration and internal forces at every step of construction can be obtained. Then, the theoretical references for every erection stage are established. This provides basic information for the erection of the bridge.

During construction, the measured items include the elevations of the girder, horizontal displacements of pylons, stresses in the girders and pylons, cable forces, temperature and temperature gradient in the bridge, as well as volume of concrete used in each segment. And material parameters such as elasticity modulus and mass density of concrete are also measured at laboratory or on-site. Those measured values provide fundamental data for error and parameter identification as well as for construction adjustment.

Through a careful and detailed simulation, continuous monitoring, parameter identification, as well as timely adjustment, the control results seem very well. The deviations of cable tensions between actual with the designed value are within 7%. Only a few stay cables need to be adjusted and retensioned after the closure of the main girder. At each deck segment, the errors of deck elevations are within 2.0 cm (0.8 in.). Before the closure of the main span, the deviation of the elevations between the two ends of the two cantilevers is quite small (see Figure 5.15). The final profile of the girder is smooth. The inclinations of pylons agree well with the designed requirement.

References

Chen, D. W., Xiang, H. F., and Zheng, X. G. 1993. "Construction Control of PC Cable-Stayed Bridge," *Journal of Civil Engineering*, 26(1), 1–11 (in Chinese).

Chen, M. X. 2003. *Construction Technique for Cable-Stayed Bridges*, Renming Jiaotong Press, Beijing, China (in Chinese).

Fujisawa, N., Tomo, H. 1985. "Computer-Aided Cable Adjustment of Cable-Stayed Bridges," *Proceedings of IABSE*, P-92/85, IABSE Periodica 4/1985, 181–190.

Furukawa, K., Inoue K., Nakayama H., Ishido K. 1986. "Studies on the Management System of Cable-Stayed Bridges under Construction Using Multi-Objective Programming Method." *Proceedings of JSCE*, Tokyo, Japan, 374(6) (in Japanese).

Han, D. J., Su, C., Wang, W. F. 2003."Technique Process and Chief Results of Construction Control for Yamen Bridge," *Bridge Construction*, 33(1), 5–8 (in Chinese).

Hidemi O. et al. 1995. "Construction of Ikara Bridge Superstructure (PC Cable-Stayed Bridge)," *Bridge and Foundation*, No. 11, pp. 7–14 (in Japanese).

Kasuga, A., Arai, H., Breen, J.E., Furukawa, K.1995. "Optimum Cable-Force Adjustment in Concrete Cable-Stayed Bridges," *Journal of Structural Engineering, ASCE*, 121(4), 685–694.

Li, Q., Bu. Y. Z., Zhang, Q. H. 2009. *Technique and Application of Geometric Control for Long-Span Cable-Stayed bridges*, Xinan Jiaotong Daxue Press, Chengdu, China (in Chinese).

Liang, L. N. 2007. "General Research on Theory and Methods of Optimized Design of Internal Force of Bridge Structures," Ph.D. Dissertation, South China University of Technology, Guangzhou, China (in Chinese).

Manbe," Y., Hirahara, N., Mukasa, N., Yabuno, M. 1999. "Accuracy Control on the Construction of Tatara Bridge," *IABSE Conference*, Malmo.

Qin, S. Q. 2007. *Construction Control for Bridge: Theory and Practice for Stress-Free State Control Method*, Renming Jiaotong Press, Beijing, China (in Chinese).

Tang, M. C. 1994. "The 40-Year Evolution of Cable-Stayed Bridges," in *Proceedings of The International Symposium on Cable-Stayed Bridges*, Lin Yuanpei et al. (Editors), pp. 30–11, Shanghai, China.

Walther, R., Houriet, B., Lsler, W., Moia, P. 1996. *Cable-Stayed Bridges* (third version), Thomas Telford, London, UK.

Wang, X. W. et al. 1996. "A Study on Determination of Cable Tensionsunder Dead Loads," *Bridge Construction*, 26(4), 1–5 (in Chinese).

Xiao, R. C., Lin, P. 1993. "Application of Computational Structural Mechanics in Construction Design and Controls of Bridge Structures," *Computational Structural Mechanics and Application*, 10(1), 92–98 (in Chinese).

Yan, D., Li, X. W., Liu, G. D. 2003."A Step Computation for the Reasonable Completion Configuration of a PC Cable-Stayed Bridge," *Journal of Chinese Highway*, 16(1), 43–46 (in Chinese).

Yan, Q. S., Han, D. J. 2001."Construction Control for the Panyu Bridge," *Journal of South China University of Technology*, 29(10), 93–097.

Zhang, X. G., Chen, A. R. 2010. *Design and Structural Performance of the Sutong Bridge*, Renmin Jiaotong Press, Beijing, China (in Chinese).

6

Substructures of Major Overwater Bridges

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6.1 Introduction

The design and construction of piers for overwater bridges present a series of demanding criteria. In service, the pier must be able to support dead and live loads successfully while resisting environmental forces such as current, wind, wave, sea ice, and unbalanced soil loads, sometimes even including downslope rockfall. Earthquake loadings present a major challenge to design, with cyclic reversing motions being propagated up through the soil and the pier to excite the superstructure. Accidental forces must also be resisted. Collision by barges and ships is becoming an increasingly serious hazard for bridge piers in waterways, For vessel collision design of bridge piers, see Chapter 4, *Bridge Engineering Handbook, Second Edition: Substructure Design*.

Soil–structure foundation interaction controls the design for dynamic and impact forces. The interaction with the superstructure is determined by the flexibility of the entire structural system and its surrounding soil.

Rigid systems attract very high forces: under earthquakes, design forces may reach 1.0*g*, whereas flexible structures, developing much less force over longer periods, are subject to a greater deflection drift. The design must endeavor to obtain an optimal balance between these two responses. The potential for scour due to currents, amplified by vortices, must be considered and preventive measures instituted.

Constructibility is of great importance, in many cases determining the feasibility of constructing a structure. During construction, temporary and permanent structures are subject to the same environmental and accidental loadings as the permanent pier, although for a shorter period of exposure and, in most cases, limited to a favorable time of the year, the so-called weather window. The construction processes employed must therefore be practicable in terms of attainment and completion. Tolerances must be a suitable compromise between practicability and future performance. Methods adopted must not diminish the future interactive behavior of the soil–structure system.

The design loadings for overwater piers are generally divided into two limit states: One is the limit state for loadings with a high probability of occurrence, for which the response should be essentially elastic. Durability needs to be considered in this limit state, primarily with respect to corrosion of exposed and embedded steel. Fatigue is not normally a factor for the pier concepts usually considered, although it does enter into the considerations for supplementary elements such as fender systems and temporary structures such as dolphins if they are utilized under conditions of cyclic loading such as waves. In seismic areas, moderate-level earthquakes, for example, those with a return period of 300–500 years, also need to be considered.

The second limit state is that of low-probability events, often termed the "safety" or "extreme" limit state. This should include earthquakes of long return periods (1000 to 3000 years) and ship collisions by major vessels. For these, a ductile response is generally acceptable, extending the behavior of the structural elements into the plastic range. Deformability is essential to absorb these high-energy loads; so some damage may be suffered with the provisions that collapse and loss of life are prevented and, usually, that the bridge can be restored to service within a reasonable time.

Plastic hinging has been adopted as a principle for this limit state on many modern structures, designed so that it will occur at a known location where it can be most easily inspected and repaired. Redundant load paths are desirable: these are usually only practicable by the use of multiple piles.

Bridge piers for overwater bridges typically represent 30%–40% of the overall cost of a bridge. In cases of deep water, they may even reach above 50%. Therefore, they deserve a thorough design effort to attain the optimum concept and details.

Construction of overwater bridge piers has an unfortunate history of delays, accidents, and even catastrophes. Many construction claims and overruns in cost and time relate to the construction of piers. Constructibility is thus a primary consideration.

The most common types of piers and their construction are described in this Chapter.

6.2 Large-Diameter Tubular Piles

6.2.1 Description

Construction of steel platforms for offshore petroleum production as well as deepwater terminals for very large vessels carrying crude oil, iron, and coal requires the development of piling with high axial and lateral capacities, which can be installed in a wide variety of soils, from soft sediments to rock. Lateral forces from waves, currents, floating ice, and earthquakes, as well as from berthing, dominate such designs. Only large-diameter steel tubular piles have proved able to meet these criteria (Figures 6.1 and 6.2).

Such large piling, ranging from 1 to 3 m in diameter and up to over 100 m in length, requires the concurrent development of very-high-energy pile-driving hammers of an order of magnitude higher

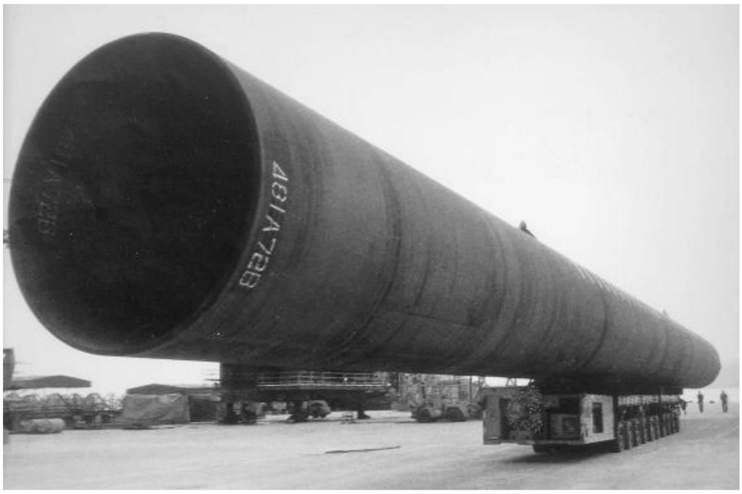

FIGURE 6.1 Large-diameter steel tubular pile for Jamuna River Bridge, Bangladesh.

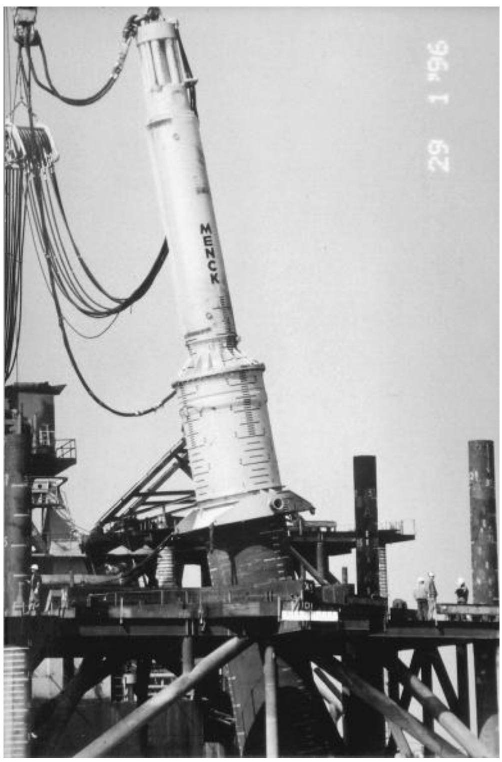

FIGURE 6.2 Driving a large-diameter steel tubular pile.

than those currently available. Drilling equipment powerful enough to drill large-diameter sockets in bedrock must also be developed (Figure 6.3).

Thus, when bridge piers were required in deeper water, with deep sediments of varying degrees or, alternatively, bare rock, where ductile responses to the lateral forces associated with earthquakes, ice, and ship impacts came to be of equal or greater importance than support of axial loads, it was only natural that technology from the offshore platform industry moved to the bridge field.

FIGURE 6.3 A steel tubular pile being installed from a jack-up barge: the socket will be drilled into rock and the entire pile filled with tremie concrete.

The results of this "lateral" transfer exceeded expectations in that it made it practicable and economical to build piers in deep waters and deep sediments where previously only highly expensive and time-consuming solutions were available.

6.2.2 Offshore Structure Practice

Design and construction practices generally follow the Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms published by the American Petroleum Institute (API-RP2A) (API, 2011). This recommended practice is revised frequently, so the latest edition should always be used. Gerwick (1986) presents the design and construction from the construction contractor's point of view.

There are many variables that affect the designs of steel tubular piles: diameter, wall thickness (which may vary over the length), penetration, tip details, pile head details, spacing, number of piles, geometry, and steel properties. The installation method and its effect on soil–pile interaction must be considered. In special cases, the tubular piles may be inclined, that is, "raked" on an angle from vertical.

In offshore practice piles are almost never filled with concrete, whereas for bridge piers the designer's unwillingness to rely solely on skin friction for support over a 100-year life, as well as his or her concern about corrosion, has led to the practice of cleaning out and filling piles with reinforced concrete. A recent advance has been to utilize the steel shell along with the concrete infill in composite action to increase strength and stiffness. The concrete infill is also utilized to resist local buckling under overload and extreme conditions. The recent practice is to fill concrete in zones of high moment.

Tubular piles are used to transfer the superimposed axial and lateral loads and moments to the soil. Under earthquakes, the soil imparts dynamic motions to the pile and hence to the structure. These interactions are highly nonlinear. To make matters even more complex, soils are typically nonuniform throughout their depth and have different values of strength and modulus.

In design, axial loads control the penetration of a pile, whereas lateral load transfer to the soil determines the pile diameter. Combined pile stresses and installation stresses determine the wall thickness. The interaction of the pile with the soil is determined by pile stiffness and diameter and leads to the development of a *P*–*y* curve, *P* being the lateral shear at the head of the pile and *y* the deflection along the pile. Although the actual behavior is very complex and can only be adequately solved by a computerized final design, an initial approximation of three diameters can give an assumed "point of fixity" about which the top of the pile bends.

Experience and laboratory tests show that the deflection profile of a typical pile in soft sediments has a first point of zero deflection about three diameters below the mudline, followed by a deflection in reverse bending and finally a second point of zero displacement. Piles driven to a tip elevation at or below this second point have generally been found to develop a stable behavior in lateral displacement even under multiple cycles of high loading.

If deflection under extreme load is significant, P-Δ effects must also be considered. Bridge piers must not only have adequate ultimate strength to resist extreme lateral loads but also limit the displacement to acceptable values. If the displacement is too great, the P-Δ effect will cause large additional bending moments in the pile and consequently additional deflection.

The axial compressive behavior of piles in bridge piers is of high importance. Settlement of the pile under service and extreme loads must be limited. The compressive axial load is resisted by skin friction along the periphery of the pile, end bearing under the steel pile tip, and end bearing of the soil plug in the pile tip. The latter must not exceed the skin friction of the soil on the inside of the pile, since otherwise the plug will slide upward. The actual characteristics of the soil plug are greatly affected by the installation procedures and are discussed in detail in Section 6.2.6.

Axial tension due to uplift under extreme loads such as earthquakes is resisted by skin friction on the periphery and the deadweight of the pile and the footing block.

Pile group action usually differs from the summation of actions of individual piles and is influenced by the stiffness of the footing block as well as by the applied bending moments and shears. This group action and its interaction with the soil are important in the final design, especially for dynamic loading such as earthquakes.

API-RP2A section G gives a design procedure for driven steel tubular piles as well as for drilled and grouted piles.

Corrosion and abrasion must be considered in determining pile wall thickness. Corrosion typically is most severe from just below the waterline to just above the wave splash level at high tide, although another vulnerable location is the mudline due to the oxygen gradient. Abrasion typically is most severe at the mudline because of moving sands, although suspended silt may cause abrasion throughout the water column.

Considering the design lifetime of a major bridge of 100 years or more, coatings are appropriate in the splash zone and above and sacrificial anodes may be used in the water column and at the mudline. Additional pile wall thickness may serve as sacrificial steel: for the seawater environment, 10–12 mm is often added.

6.2.3 Steel Pile Design and Fabrication

Tubular steel piles are typically fabricated from steel plates, rolled into "cans" with the longitudinal seam being automatically welded. These cans are then joined by circumferential welds. Obviously, these welds are critical to the successful performance of the piles. During installation by pile hammer, the welds are often stressed very highly under repeated blows: defective welds may crack in the welds or the heat-affected zone. Welds should achieve as much full joint penetration as practicable, and the external weld profile should merge smoothly with the base metal on either side.

API-RP2A section L gives guidance on fabrication and welding. Fabricated piles should meet specified tolerances for both pile straightness and cross-sectional dimensions at the ends. These control the average diameter and out-of-roundness of the pile. Out-of-roundness is of special concern as it affects the ability to match adjacent sections for welding.

Inspection recommendations are given in API-RP2A section N. Table N.4-1 in API-RP2A section N, with reference to structural tubulars, calls for 10% of the longitudinal seams to be verified by either ultrasonic testing (UT) or radiography (RT). For the circumferential weld seams and the critical intersection of the longitudinal and circumferential seams, 100% UT or RT is required.

Because of the typically high stresses to which piles supporting bridge piers are subjected, both under extreme loads and during installation, as well as the need for weldability of relatively thick plates, it is common to use a fine-grained steel of 290–350 MPa yield strength for tubular piles.

Pile wall thickness is determined by a number of factors. The thickness may be varied along the length, being controlled at any specific location by the loading conditions during service and during installation.

A typical pile used for a bridge pier is fixed at the head. Hence, the maximum combined bending and axial loads will occur within the one and a half diameters immediately below the bottom of the footing. Local buckling may occur. Repeated reversals of bending under earthquakes may even lead to fracture. This area is therefore generally made of thicker steel plates. Filling with concrete will prevent local buckling. General column buckling also needs to be checked and will usually be a maximum at a short distance below the mudline.

Installation may control the minimum wall thickness. The hammer blows develop high compressive waves that travel down the pile, reflecting from the tip in amplified compression when high tip resistance is encountered. When sustained hard driving with large hammers is anticipated, the minimum pile wall thickness should be $t = 6.35 + D/100$, where t and D are in millimeters. The drivability of a tubular pile is enhanced by increasing the wall thickness. This reduces the time of driving and enables greater penetration.

During installation, the weight of the hammer and appurtenances may cause excessive bending if the pile is being installed on a batter. Hydraulic hammers are usually fully supported on the pile, whereas steam hammers and diesel hammers are partially supported by the crane.

If the pile is cleaned out during driving to enable the desired penetration, external soil pressures may develop high circumferential compression stresses. These interact with axial driving stresses and may lead to local buckling.

The tip of the pile is subject to very high stresses, especially if the pile encounters boulders or must be seated in rock. This may lead to distortion of the tip, which is then amplified during successive blows. In extreme cases, the tip may "tear" or may "accordion" in a series of short local axial buckles. Cast steel driving shoes may be employed in such cases; they are usually made of steels of high toughness as well as high yield strength. The pile head also must be thick enough to withstand both the local buckling and the bursting stresses due to Poisson's effect.

The transition between sections of different pile wall thickness must be carefully detailed. In general, the change in thickness should not be more than 12 mm at a splice and the thicker section should be beveled on a slope of 1:4.

6.2.4 Transportation and Upending of Piles

Tubular piles may be transported by barges. For loading, they are often simply rolled onto the barges and then blocked and chained down. They may also be transported by self-flotation. The ends are bulkheaded during deployment. The removal of bulkheads can impose serious risks if it is not carefully planned. One end should be lifted above water for the removal of that bulkhead, then the other. If one bulkhead is to be removed underwater by a diver, the water inside must first be equalized with the outside water; otherwise, the rush of water will suck the diver into the pipe. Upending will produce high bending moments, which limit the length of sections of a long pile (Figure 6.4). Otherwise, the pile may be buckled.

6.2.5 Driving of Piles

The driving of large-diameter tubular piles (Gerwick, 1986) is usually done by a very large pile hammer. The required size can be determined by both experience and a drivability analysis, which incorporates the soil parameters.

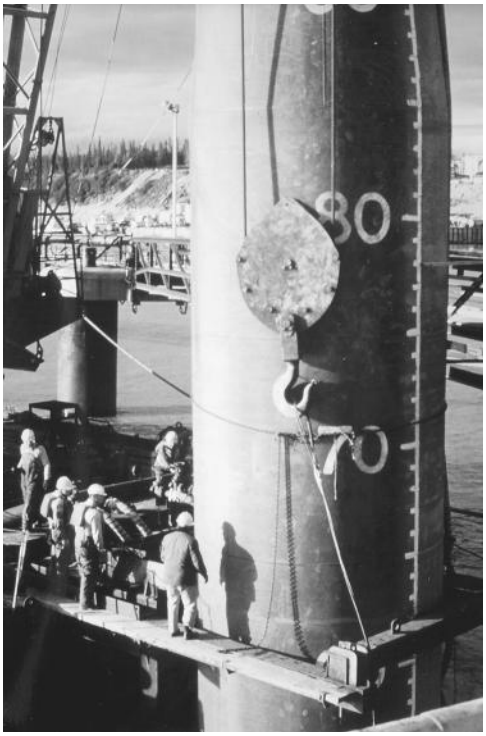

FIGURE 6.4 A large-diameter tubular steel pile being positioned.

Frequently, the tubular pile for a bridge pier is too long or too heavy to install as a single section. Hence, piles must be spliced during driving. To assist in splicing, stabbing guides may be preattached to the tip of the upper segment, along with a backup plate. The tip of the upper segment should be prebeveled for welding.

Splicing is time consuming. Fortunately, on a large-diameter pile of 2–4 m diameter, there is usually space to work two to three crews concurrently. Weld times of 4–8 hours may be required. Then the pile must cool down (for typically 2 hours), and nondestructive testing must be performed. Following this, the hammer must be repositioned on top of the pile. Thus, the total elapsed time may be 9–12 hours, during which the skin friction on the pile sides "sets up," increasing the driving resistance and typically requiring a number of blows to break the pile loose and resume penetration.

When very high resistance is encountered, various methods may be employed to reduce the resistance so that the design pile tip may be reached. Care must be taken that these aids do not lessen the capacity of the pile to resist its design loads.

High resistance of the tubular pile is primarily due to plugging of the tip; the soil in the tip becomes compacted and the pile behaves as a displacement pile instead of cutting through the soil. The following steps may be employed:

1. Jetting internally to break up the plug, but not below the tip: the water level inside must be controlled, that is, not allowed to build up much above the outside water level, to prevent from piping underneath. Although a free jet or arrangement of jets may be used, a very effective method is to manifold a series of jets around the circumference and weld the down-going pipes to the shell (Figure 6.5). Note that these pipes will pick up parasitic stresses under pile hammer blows.
2. Cleanout by airlift: this is common practice when using large-diameter tubular piles for bridge piers but has serious risks associated with it. The danger arises from the fact that an airlift can remove water very rapidly from a pile, creating an unbalanced head at the tip and allowing run-in

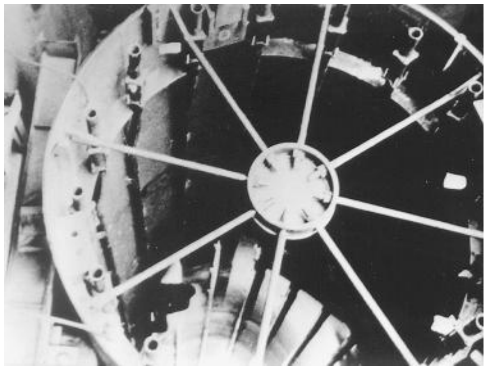

FIGURE 6.5 The arrangement of internal jet piping and "spider" struts in a large-diameter tubular pile.

of soil. Such a run-in can result in major loss of resistance, not only under the tip in end bearing but also along the sides in skin friction.

Unfortunately, this problem has occurred in a number of projects. The preventive method is to have a pump refilling the pile at the same rate that the airlift empties it—a very difficult matter to control. If structural considerations allow, a hole can be cut in the pile wall so that the water is always automatically balanced. This, of course, will be effective only when the hole is below water. The stress concentrations around such a hole need to be carefully evaluated. Because of the associated risks and the service consequences of errors in field control, the use of an airlift is often prohibited. The alternative method, one that is much safer, is the use of a grab bucket (orange-peel bucket) to remove the soil mechanically. Then, the water level can be controlled with relative ease.

3. Drilling ahead a pilot hole, using slurry: if the pile is kept full of slurry to the same level as the external water surface, then a pilot hole, not exceeding 75% of the diameter, may be drilled ahead from one to two diameters. Centralizers should be used to keep the drilled hole properly aligned. Either bentonite or a polymer synthetic slurry may be used. In soils such as stiff clay or where a binder prevents sloughing, seawater may be used. Reverse circulation is important to prevent erosion of soils due to high-velocity flows. Drilling ahead is typically alternated with driving. The final seating should be by driving beyond the tip of the drilled hole to remobilize plug resistance.
4. External jetting: external jetting relieves the skin friction during driving but sometimes permanently reduces both lateral and axial capacities. Further, it is of only secondary benefit compared with internal jetting to break up the plug. In special cases, it may still be used. The only practicable method to use with long and large tubular piles is to weld the piping on the outside or inside with holes through the pile wall. Thus, the external jetting resembles the one used on much larger open caissons. As with them, a low-pressure, high-volume water flow is most effective in reducing skin friction. After penetration to the tip, grout may be injected to partially restore lateral and axial capacities.

6.2.6 Utilization of Piles in Bridge Piers

There are several possible arrangements for tubular piles when used for bridge piers. These differ in some cases from those used in offshore platforms:

1. The pile may be driven to the required penetration and left with the natural soil inside. The upper portion may then be left with water fill or, in some cases, purposely left empty to reduce mass and weight; in this case, it must be sealed by a tremie concrete plug. To ensure full bonding with the inside wall, that zone must be thoroughly cleaned by wire brush on a drill stem or by jet.

   For piles fixed at their head, at least two diameters below the footing are filled with concrete to resist local buckling. Studs are installed in this zone to ensure shear transfer.

2. The pile, after being driven to final penetration, is cleaned out to within one diameter of the tip. The inside walls are cleaned by a wire brush or jet. A cage of reinforcing steel may be placed to augment the bending strength of the tubular shell. Centralizers should be used to ensure accurate positioning. The pile is then filled with tremie concrete. Alternatively, an insert steel tubular with a plugged tip may be installed with centralizers and the annular space filled with tremie grout. The insert tubular may need to be temporarily weighted and/or held down to prevent flotation in the grout.

   Complete filling of a tubular pile with concrete is not always warranted. The heat of hydration is a potential problem, requiring special concrete mix designs and perhaps precooling.

   The reasons for carrying out this practice, which is very often adopted for bridge piers although seldom used in offshore structures, are as follows:

   a. Concern over corrosion loss of the steel shell over a 100-year lifetime
   b. A need to positively ensure the ability of the permanent plug to sustain end bearing
   c. Prevention of local buckling near the mudline and at the pile head
   d. To obtain the benefits of composite behavior in stiffness and bending capacity

   If no internal supplemental reinforcement is required, then the benefits of b, c, and d in the aforementioned list may be achieved by simply filling with tremie concrete. To offset the heat of hydration, the core may be placed as precast concrete blocks, subsequently grouted into monolithic behavior. Alternatively, only the annulus is completely filled. The insert pile is left empty except at the head and the tip.

   The act of cleaning out the pile close to its tip inevitably causes stress relaxation in the soil plug below the cleanout. This will mean that under extreme axial compression the pile will undergo a small settlement before it restores its full resistance. To prevent this, after the concrete plug has hardened grout may be injected just beneath the plug, at a pressure that restores the compactness of the soil but is not so great as to pipe under the tip or fracture the foundation, or the pile may be reseated by driving.
3. The tubular pile, after being installed to design penetration, may be filled with sand up to two diameters below the head and then with tremie concrete to the head. Reinforcing steel may be placed in the concrete to transfer part of the moment and tension into the footing block. Studs may be preinstalled on that zone of the pile to ensure full shear transfer. The soil and the sand plug will act to limit local buckling at the mudline under extreme loads.
4. A socket may be drilled into rock or the hard material beyond the tip of the driven pile and then filled with concrete. Slurry is used to prevent degradation of the surface of the hole and sloughing. Seawater may be used in some rocks, but it may cause slaking in others such as shale and siltstone. Bentonite slurry coats the surface of the hole; the hole should be flushed with seawater just before concreting. Synthetic slurries are best since they react in the presence of calcium ions from the concrete to improve the bond. Synthetic polymer slurries biodegrade and thus may be environmentally acceptable for discharge into the water.

When a tubular pile is seated on rock and the socket is then drilled below the tip of the pile, it often is difficult to prevent the run-in of sands from around the tip and to maintain proper circulation. Therefore, after landing, a hole may be drilled a short distance, for example, with a churn drill or a down-the-hole drill, and then the pile reseated by the pile hammer.

Either insert tubulars or reinforcing steel cages are placed in the socket, extending well up into the pile. Tremie concrete is then placed to transfer the load in shear. In the case where a tubular insert pile is used, its tip may be plugged. Then, grout may be injected into the annular space to transfer the shear.

Grout should not be used to fill sockets of large-diameter tubulars. The heat of hydration will damage the grout, reducing its strength. Tremie concrete should be used instead, employing small-size coarse aggregates, for example, 15 mm, to ensure workability and flowability.

Although most sockets for offshore bridge piers are cylindrical extensions of the tubular pile, in some offshore oil platforms belled footings have been constructed to transfer the load in end bearing. Hydraulically operated belling tools are attached to the drill string. Whenever transfer in end bearing is the primary mechanism, the bottom of the hole must be cleaned of silt just prior to the placement of concrete.

6.2.7 Prestressed Concrete Cylinder Piles

As an alternative to steel tubular piling, prestressed concrete cylinder piles have been used for a number of major overwater bridges, from the San Diego–Coronado and Dunbarton Bridges in California to bridges across Chesapeake Bay and the Yokohama cable-stayed bridge (Figures 6.6 and 6.7). Diameters range from 1.5 to 6 m and more. They offer the advantage of durability and high axial compressive capacity. To counter several factors producing circumferential strains, especially thermal strains, spiral reinforcement of adequate cross-sectional area is required. This spiral reinforcement should be closely spaced in the 2 m zone just below the pile cap, where sharp reverse bending occurs under lateral loading.

Pile installation methods vary from driving and jetting of smaller diameter piles to drilling in large-diameter pilings (Figure 6.8).

6.2.8 Footing Blocks

The footing block constructed at the top of large-diameter tubular piles serves the purpose of transmitting forces from the pier shaft to the piles. Hence, it is subjected to large shears and significant moments. The shears require extensive vertical reinforcement, for both global shears (from the pier shaft) and local

FIGURE 6.6 A large-diameter prestressed concrete pile for Napa River Bridge, California.

FIGURE 6.7 A prestressed concrete cylinder pile for Oosterschelde Bridge, the Netherlands.

FIGURE 6.8 Installing a concrete cylinder pile by internal excavation, jetting, and pull-down force from a barge.

shears (punching shear from the piles). Large concentrations of reinforcement are required to distribute the moments. Posttensioned tendons may be effectively utilized.

Primary forces typically produce compression in the upper surface of the footing block, and secondary forces and particularly high temporary stresses caused by the heat of hydration produce tension in the top surface. Thus, adequate horizontal steel must be provided at the top and bottom in both directions.

The heat of hydration of the cemetitious materials in a large footing block develops over a period of several days. Due to the mass of the block, the heat in the core may not dissipate and return to the ambient for several weeks.

In the meantime, the outside surface cools and contracts, producing tension, which often leads to cracking. Where inadequate reinforcement is provided, the steel may stretch beyond yield so that the cracks become permanent. If proper amounts of reinforcement are provided, then the cracking that develops will be well distributed, individual cracks will remain small, and the elastic stress in the reinforcement will tend to close the cracks as the core cools.

Internal laminar cracking may also occur, so vertical reinforcement and middepth reinforcement should also be considered.

Footing blocks may be constructed in place, just above water, with precast concrete skirts extending down below low water level to prevent small boats and debris from being trapped below. In this case, the top of the piles may be exposed at low water, requiring special attention on the prevention of corrosion.

Footing blocks may be constructed below water. Although cofferdams may be employed, the most efficient and economical way is usually to prefabricate the shell of the footing block. This is then floated into place. Corner piles are then inserted through the structure and driven to grade. The prefabricated box is then lowered down by ballasting, supported and guided by the corner piles. Then the remaining piles are threaded through holes in the box and driven. Final connections are made by tremie concrete.

Obviously, there are variations of the aforementioned procedure. In some cases, portions of the box are kept permanently empty, utilizing their buoyancy to offset part of the deadweight.

The transfer of forces into the footing block requires careful detailing. It is usually quite difficult to transfer full moments by means of reinforcing inside the pile shell. If the pile head can be dewatered, reinforcing steel bars can be welded to the inside of the shell. Cages set in the concrete plug at the head may employ bundled bars with mechanical heads at their top. Alternatively, the pile may be extended up through the footing block. Shear keys can be used to transfer shear. Posttensioning tendons may run through and around the pile head.

6.3 Cofferdams for Bridge Piers

6.3.1 Description

The word "cofferdam" is a very broad term to describe a construction that enables an underwater site to be dewatered. As such, cofferdams can be large or small. Medium-sized cofferdams of horizontal dimensions from 10 to 50 m have been widely used to construct the foundations of bridge piers in water and soft sediments up to 20 m in depth; a few have been larger and deeper (Figure 6.9). Typical bridge pier cofferdams are constructed of steel sheet piles supported against external pressures by internal bracing.

A few very large bridge piers, such as anchorages for suspension bridges, utilize a ring of self-supporting sheet pile cells. The interior is then dewatered and excavated to the required depth. A recent such development has been the construction of a circular ring wall of concrete by the slurry trench method (Figures 6.10 and 6.11). Concrete cofferdams have also used a ring wall of precast concrete sheet piles or even cribs.

6.3.2 Design Requirements

Cofferdams must be designed to resist the external pressures of water and soil (Gerwick and Durnal, 2012). If, as is usual, a portion of the external pressures is designed to be resisted by the internal passive pressure of the soil, the depth of penetration must be selected conservatively, taking into account potential sudden reductions in passive pressure due to water flow beneath the tip as a result of unbalanced water pressures or jetting of piles. The cofferdam structure itself must have adequate vertical support for self-loading and necessary equipment under all conditions.

FIGURE 6.9 Large steel sheet pile cofferdam for Second Delaware Memorial Bridge, showing bracing frames.

FIGURE 6.10 Slurry wall cofferdam for Kawasaki Island ventilation shaft, Trans-Tokyo Bay Tunnels and Bridge.

FIGURE 6.11 Concrete ring wall cofferdam constructed by slurry trench methods.

In addition to primary design loads, other loading conditions and scenarios include currents and waves, debris and ice, overtopping by high tides, floods, or storm surges. Although earthquake-induced loads, acting on the hydrodynamic mass, have generally been neglected in the past, they are now often being considered in major cofferdams, taking into account the lower input accelerations appropriate for the reduced time of exposure and, where appropriate, the reduced consequences.

Operating loads due to the mooring of barges and other floating equipment need to be considered alongside. The potential for scour must be evaluated, along with appropriate measures to reduce the scour. When the cofferdam is located on a sloping bank, the unbalanced soil loads need to be properly resisted. Accidental loads include impact from boats and barges, especially those working around the site.

The cofferdam as a whole must be adequately supported against the lateral forces of current waves, ice, and moored equipment, as well as unbalanced soil loads. Although a large deepwater cofferdam appears to be a rugged structure, when fully excavated and prior to placement of the tremie concrete seal, it may be too weak to resist global lateral forces. Large tubular piles, acting as spuds in conjunction with the space frame or batter piles, may be needed to provide stability.

The cofferdam design must integrate the piling and footing block properly. For example, sheet piles may prevent the installation of batter piles around the periphery. To achieve adequate penetration of sheet piles and to accommodate batter piles, the cofferdam may need to be enlarged. The arrangement of the bracing should facilitate any subsequent pile installation.

To enable dewatering of the cofferdam (Figure 6.12), a concrete seal is constructed, usually by the tremie method. This seal is designed to resist the hydrostatic pressure by its own buoyant weight and by uplift resistance provided by the piling, the latter being transferred to the concrete seal course by shear (Figure 6.13).

In shallow cofferdams, a filter layer of coarse sand and rock may permit pumping without a seal. However, in most cases, a concrete seal is required. In some recent construction, a reinforced concrete footing block was designed to be constructed underwater to eliminate the need for a separate concrete seal. In a few cases, a drainage course of stone is placed below the concrete seal; it is then kept dewatered to reduce the uplift pressure. Emergency relief pipes through the seal course will prevent structural failure of the seal in case the dewatering system fails.

The underwater lateral pressure of the fresh concrete in the seal course and footing block must be resisted by external backfill against the sheet piles or by internal ties.

6.3.3 Internally Braced Cofferdams

These are the predominant type of cofferdams. They are usually rectangular in shape, to accommodate a regular pattern of cross-lot bracing.

The external wall is composed of steel sheet piles of appropriate section modulus to develop bending resistance. The loading is then distributed by horizontal wales to cross-lot struts. These struts should be laid out on a plan, which will permit excavation between them, to facilitate the driving of piling and to eliminate, as far as practicable, the penetration of bracing through the permanent structure.

Wales are continuous beams loaded by the uniform bearing of sheet piles against them. They are also loaded axially in compression when they serve as a strut to resist the lateral loads acting on them endwise. Wales, in turn, deliver their normal loads to the struts, developing concentrated local bearing loads superimposed on the high bending moments, tending to produce local buckling. Stiffeners are generally required.

Although stiffeners are readily installed on the upper side, they are difficult to install on the underside and difficult to inspect. Hence, these stiffeners should be preinstalled during fabrication of the members.

The wales are restrained from global buckling in the horizontal plane by the struts. In the vertical plane, they are restrained by the friction of the sheet piles, which may need to be supplemented by direct fixation. Blocking of timber or steel shims is installed between the wales and sheet piles to fit irregularities in sheet pile installation and to fill in the needed physical clearances.

FIGURE 6.12 Dewatering the cofferdam for the main tower pier of Second Delaware Memorial Bridge.

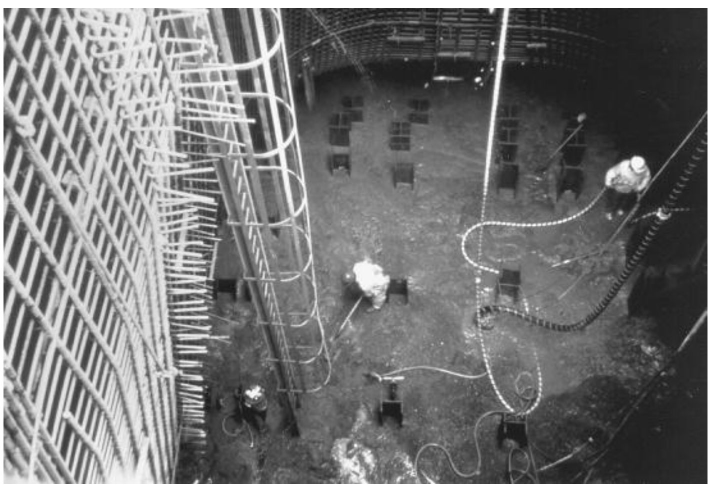

FIGURE 6.13 Pumped-out cofferdam showing tremie concrete seal and predriven steel H-piles.

Struts are horizontal columns subject to high axial loads, as well as vertical loads from self-weight and any equipment that is supported by them. Their critical concern is stability against buckling. This is countered in the horizontal plane by intersecting struts but usually needs additional support in the vertical plane, either by piling or by trussing two or more levels of bracing.

Orthogonal horizontal bracing may be all at one elevation, in which case the intersections of the struts have to be accommodated, or they may be vertically offset, with one level resting on top of the other. The latter is normally easier since otherwise the intersections must be detailed to transmit the full loads across the joint. This is particularly difficult if struts are made of tubular pipe sections. If struts are made of wide-flanged or H-section members, then it will usually be found preferable to construct them with the weak axis in the vertical plane, facilitating the detailing of strut to strut intersections as well as strut to wale intersections. In any event, stiffeners are required to prevent the buckling of flanges.

For deepwater piers, cofferdam bracing is best constructed as a space frame, with two or more levels joined together by posts and diagonals in the vertical plane. This space frame may be completely prefabricated and set as a unit, supported by vertical piles. These supporting piles are typically of large-diameter tubular members, driven through sleeves in the bracing frame and connected to it by blocking and welding.

The setting of such a space frame requires a very large crane barge or equivalent, with both adequate hoisting capacity and sufficient reach. Sometimes, therefore, the bracing frame is made buoyant to be partially or wholly self-floating. Tubular struts can be kept empty, and supplemental buoyancy can be provided by pontoons.

Another way to construct the bracing frame is to erect one level at a time, supported by large tubular piles in sleeves. The lower level is first erected, followed by the posts and diagonal bracing in the vertical plane. The lower level is then lowered by hoists or jacks so that the second level can be constructed just above water and connections made in the dry.

A third way is to float in the prefabricated bracing frame on a barge, drive spud piles through sleeves at the four corners, and hang the bracing frame from the piles. Then the barge is floated out at low tide and the bracing frame lowered to position.

6.3.4 Circular Cofferdams

Circular cofferdams are also employed, with ring wales to resist the lateral forces in compression. Their dimensions are large, and their ring compression is high. Unequal loading is frequently due to differential soil pressures. Bending moments are very critical, since they add to the compression on one side. Thus, the ring bracing must have substantial strength against buckling in the horizontal plane.

6.3.5 Excavation

Excavation should be carried out in advance of setting the bracing frame or sheet piles, whenever practicable. Although due to side slopes the total volume of excavation will be substantially increased, the work can be carried out more efficiently and rapidly than excavation within a bracing system.

When an open-cut excavation is not practicable, excavation must be carried out by working through the bracing with a clamshell bucket. Struts should be spaced as widely as possible to permit the use of a large bucket. Care must be taken to prevent impact with the bracing while the bucket is being lowered and from snagging the bracing from underneath while the bucket is being hoisted. These accidental loads may be largely prevented by temporarily standing up sheet piles against the bracing in the well being excavated to act as guides for the bucket.

Except when the footing course is constructed directly on a hard stratum or rock, over excavation by 1 m or so is usually found beneficial. Then the over excavation can be backfilled to grade by crushed rock.

6.3.6 Driving of Piles in Cofferdams

Pilings can be driven before the bracing frame and sheet piles are set. They can be driven by underwater hammers or followers. To ensure proper location, the pile driver must be equipped with telescopic leads or a template must be set on the excavated river bottom or seafloor.

Piling may alternatively be driven after the cofferdam is installed, using the bracing frame as a template. In this case, an underwater hammer presents problems of clearance due to its large size, especially for batter piles. Followers may be used, or often, more efficiently, the piles may be lengthened by splicing to temporarily extend them all the way to above the water. They are then cut off to grade after the cofferdam is dewatered. This procedure obviates the problems occasioned if a pile fails to develop proper bearing since underwater splices are not needed. It also eliminates cutoff waste. The long sections of piling cutoff after dewatering can be taken back to the fabrication yard and respliced for use on a subsequent pier.

All the aforementioned aspects assume driven steel piling, which is the prevalent type. However, in several recent projects drilled shafts have been constructed after the cofferdam was excavated. In the latter case, a casing must be provided, seated sufficiently deep into the bottom soil to prevent run-in or blowout (see Section 6.2.6, item 4).

Driven timber or concrete piles may also be employed, typically using a follower to drive them below water.

6.3.7 Tremie Concrete Seal

The tremie concrete seal course functions to resist hydrostatic uplift forces to permit dewatering. As described in Section 6.3.2, it usually is locked to the foundation piling to anchor the slab. It may be reinforced to enable it to distribute the pile loads and to resist cracking due to heat of hydration.

"Tremie" concrete is a term derived from the French to designate concrete placed through a pipe. The term has subsequently evolved to incorporate both a concrete mix and a placement procedure. Underwater concreting has had both significant successes and significant failures. Yet the system is inherently reliable, and concrete equal to or better than concrete placed in the dry has been produced at depths of up to 250 m. The failures have led to large cost overruns due to required corrective actions. They have largely been due to inadvertently allowing the concrete to flow through or be mixed with the water, which causes washout of the cement and segregation of aggregates.

Partial washout of cement leads to the formation of a surface layer of laitance, which is a weak paste. This may harden after a period of time into a brittle chalklike substance.

The tremie concrete mix must have an adequate quantity of cementitious materials. These can be a mixture of Portland cement with either fly ash or blast furnace slag (BFS). These are typically proportioned so as to reduce the heat of hydration and promote cohesiveness. A total content of cementitious materials of 400 kg/m^3 (~700 lb/cy) is appropriate for most cases.

Aggregates are preferably rounded gravel, so they flow more readily. However, crushed coarse aggregates may be used if an adequate content of sand is provided. The gradation of the combined aggregates should be heavy toward the sand portion—a 45% sand content appears optimum for proper flow. The maximum size of coarse aggregates should be kept small enough for them to flow smoothly through the tremie pipe and overcome any restrictions such as those caused by reinforcement. The use of coarse aggregates of 20 mm maximum size appears optimum for most bridge piers.

A conventional water-reducing agent should be employed to keep the water/cementitious material ratio below 0.45. Superplasticizers should not normally be employed for the typical cofferdam, since workability and flowability may be lost prematurely due to the heat generated in the mass concrete. Retarders are essential to prolong the workable life of the fresh mix if superplasticizers are used.

Other admixtures are often employed. Air entrainment improves flowability at shallow water depths, but the beneficial effects are reduced at greater depths due to increased external pressure. Weight to reduce uplift is also lost.

Microsilica may be included in amounts of up to 6% of the cement to increase the cohesiveness of the mix, thus minimizing segregation. It also reduces bleed. Antiwashout admixtures (AWAs) are also employed to minimize washout of cementitious materials and segregation. They tend to promote self-leveling and flowability. Both microsilica and AWAs may require the use of superplasticizers, in which case retarders are essential. However, a combination of silica fume and AWAs should be avoided as it typically is too sticky and does not flow well.

Heat of hydration is a significant problem with the concrete seal course, as well as with the footing block, due to the mass of concrete. Therefore, the concrete mix is often precooled, for example, by chilling of the water or the use of ice. Liquid nitrogen is sometimes employed to reduce the temperature of the concrete mix to as low as 5°C. Heat of hydration may be reduced by incorporating substantial amounts of fly ash to replace an equal portion of cement. BFS cement can also be used to reduce heat, provided the BFS is not ground too fine, that is, not finer than 2500 cm^2/g, and the proportion of slag is at least 70% of the total.

The tremie concrete mix may be delivered to the placement pipe by any of several means. Pumping and conveyor belts are best because of their relatively continuous flow. The pipe for pumping should be precooled and insulated or shielded from the sun; conveyor belts should be shielded. Another means of delivery is by bucket. Buckets should be air operated to gradually feed the concrete to the hopper at the upper end of the tremie pipe. Placement down the tremie pipe should be by gravity feed only (Figure 6.14).

Although many placements of tremie concrete have been carried out by pumping, there have been serious problems in large placements such as cofferdam seals. The reasons include the following:

1. Segregation in the long down-leading pipe, partly due to the formation of a partial vacuum and partly due to the high velocity of concrete flow through the pipe
2. The high pressures at discharge
3. The surges of pumping

Since the discharge is into fresh concrete, these phenomena lead to turbulence and promote intermixing with water at the surface, forming excessive laitance.

These discharge effects can be contrasted with the smooth flow from a gravity-fed pipe in which the height of the concrete inside the tremie pipe automatically adjusts to match the external pressure of

FIGURE 6.14 Placing underwater concrete through hopper and tremie pipe for Verrazano Narrows Bridge, New York.

water versus the previously placed concrete. For piers at considerable depths, this balance point will be about halfway down. The pipe should have an adequate diameter in relation to the maximum size of coarse aggregates to permit remixing: a ratio of 8 to 1 is the minimum. A slight inclination of the tremie pipe from the vertical will slow the feed of new concrete and facilitate the escape of entrapped air.

For starting tremie concrete placement, the pipe must first be filled slightly above middepth. This is most easily done by plugging the end and placing the empty pipe on the bottom. The empty pipe must be negatively buoyant. It must also be able to withstand the external hydrostatic pressure as well as the internal pressure of underwater concrete. Joints in the tremie pipe should be gasketed and bolted to prevent water being sucked into the mix by venturi action. To commence placement, the tremie pipe, which is slightly more than half full, is raised 150 mm off the bottom. The temporary plug then comes off, and the concrete flows out. The aforementioned procedure can be used for both starting and resuming a placement, as, for example, when the tremie is relocated or after a seal has been inadvertently lost.

The tremie pipe should be kept embedded in the fresh concrete mix for a sufficient distance to provide backpressure on the flow (typically 1 m minimum), but not so deep as to become stuck in the concrete due to its initial set. This requires adjusting the retarding admixture to match the rate of concrete placement and the area of the cofferdam against the time of set, keeping in mind the acceleration of set due to heat as the concrete hydrates.

Another means for starting tremie concrete placement is to use a pig, which is forced down the pipe by the weight of the concrete, expelling the water below. This pig should be round or cylindrical, preferably the latter, and equipped with wipers to prevent leakage of grout and jamming by a piece of aggregate. An inflated ball, such as an athletic ball (a volleyball or basketball) must never be used; such balls collapse at about 8 m water depth. A pig should not be used to restart a placement, since it would force a column of water into the fresh concrete previously placed.

Mixes of the tremie concrete will flow outward on a slope of about 1 on 8 to 1 on 10. With AWAs, an even flatter surface can be obtained.

A trial batch with underwater placement in a shallow pit or tank should always be done before the actual placement of the concrete seal. This is to verify the cohesiveness and flowability of the mix. Laboratory tests are often inadequate and misleading, so a large-scale test is important. A trial batch of 2 to 3 m^3 has often been used.

Tremie concrete placement will exert outward pressure on the sheet piles, causing them to deflect. This may in turn allow new grout to run down past the already set concrete, increasing the external pressure. To offset this, the cofferdam can be partially backfilled before starting tremie concreting and tied across the top. Alternatively, dowels can be welded on the sheets to tie into the concrete as it sets; then the sheet piles have to be left in place.

Due to heat of hydration, the concrete seal will expand. Maximum temperature may not be achieved for several days. Cooling of the mass is gradual, starting from the outside, and ambient temperature may not be achieved for several weeks. Thus, the external shell cools and shrinks while the interior is still hot. This can produce severe cracking, which, if not constrained, will create permanent fractures in the seal or footing. Therefore, in the best practice reinforcing steel is placed in the seal to both provide a restraint against cracking and help pull the cracks closed as the mass cools.

After a relatively few days, the concrete seal will usually have developed sufficient strength to permit dewatering. Once exposed to the air, especially in winter, the surface concrete cools too fast and may crack. Use of insulation blankets will keep the temperature more uniform. They will, of course, have to be temporarily moved to permit subsequent work to be performed.

6.3.8 Pier Footing Block

The pier footing block is constructed next. Reinforcement is required on all faces, not only for structural response but also to counteract thermal strains. Reinforcement for the footing block is discussed in more detail in Chapter 8.

The concrete expands as it is placed in the footing block due to the heat of hydration. At this stage, it either is still fresh or, if set, has a very low modulus. Then it hardens and bonds to the tremie concrete below. The lock between the two concrete masses is made even more rigid if piling protrudes through the top of the tremie seal, which is common practice. Now the footing block cools and tries to shrink but is restrained by the previously placed concrete seal. Vertical cracks typically form. Only if there is sufficient bottom reinforcement in both directions can this shrinkage and cracking be adequately controlled. Note that these tensile stresses are permanently locked into the bottom of the footing block and the cracks will not close with time, although creep will be advantageous in reducing residual stresses.

After the footing block has hardened, blocking may be placed between it and the sheet piles. This, in turn, may permit the removal of the lower level of bracing. As an alternative to bracing, the footing block may be extended all the way to the sheet piles, using a sheet of plywood to prevent adhesion.

6.3.9 Pier Shaft

The pier shaft is then constructed. Blockouts may be required to allow the bracing to pass through. The internal bracing is removed in stages, taking care to ensure that this does not result in overloading a brace above. Each stage of removal should be evaluated.

Backfill is then placed outside the cofferdam to bring it up to the original seabed. The sheet piles can then be removed. The first sheets are typically difficult to break loose and may require driving or jacking in addition to vibration. Keeping in mind the advantage of steel sheet piles in preventing the undermining of the pier due to scour, as well as the fact that the removal of sheets always loosens the surrounding soil, hence reducing the passive lateral resistance, it is often desirable to leave the sheet piles in place below the top of the footing. They may be cut off underwater by divers; then the tops are pulled by vibratory hammers.

Antiscour stone protection is now placed, with an adequate filter course or fabric sheet in the case of fine sediments.

6.4 Open Caissons

6.4.1 Description

Open caissons have been employed for some of the largest and deepest bridge piers (O'Brien et al., 1996) These are an extension of the "wells" that have been used for some 2000 years in India. The caisson may be constructed above its final site, supported on a temporary sand island, and then sunk by dredging out within the open wells of the caisson, the deadweight acting to force the caisson down through the overlying soils (Figure 6.15). Alternatively, especially in sites overlaid by deep water, the caisson may be prefabricated in a construction basin, floated to the site by self-buoyancy, augmented as necessary by temporary floats or lifts, and then progressively lowered into the soils while building up the top.

Open caissons are effective but costly, due to the large quantity of material required and the labor for working at the overwater site. Historically, they have been the only means of penetrating through deep overlying soils onto a hard stratum or bedrock. However, their greatest problem is maintaining stability during the early phases of sinking, when they are neither afloat nor firmly embedded and supported. Long and narrow rectangular caissons are especially susceptible to tipping, whereas square and circular caissons of substantial dimensions relative to the water depth are inherently more stable. Once the caisson tips, it tends to drift off position. It is very difficult to bring it back to the vertical without overcorrecting.

When the caisson finally reaches its founding elevation, the surface of rock or hard stratum is cleaned and a thick tremie concrete base is placed. Then the top of the caisson is completed by casting a large capping block on which to build the pier shaft.

FIGURE 6.15 An open caisson positioned within steel jackets on pens.

6.4.2 Installation

The sinking of the cofferdam through the soil is resisted by skin friction along the outside and by bearing on the cutting edges. Approximate values of resistance may be obtained by multiplying the friction factor of sand on concrete or steel by the at-rest lateral force at that particular stage, $f = ØK_0wh^2$, where f is the unit frictional resistance, Ø the coefficient of friction, w the underwater unit weight of sand, K_0 the at-rest coefficient of lateral pressure, and h the depth of sand at that level. Then, f is summed over the embedded depth. In clay, cohesive shear controls the skin friction. The bearing value of the cutting edges is generally the "shallow bearing value," that is, five times the shear strength at that elevation.

These resistances must be overcome by the deadweight of the caisson structure, reduced by the buoyancy acting on the submerged portions. This deadweight may be augmented by jacking forces on ground anchors.

Skin friction is usually reduced by lubricating jets causing the upward flow of water along the sides. Compressed air may be alternated with water through the jets; bentonite slurry may be used to provide additional lubrication. The bearing on the cutting edges may be reduced by cutting jets built into the walls of the caisson or by free jets operating through holes formed in the walls. Finally, the vibration of soils near and around the caisson may help to reduce the frictional resistance.

When a prefabricated caisson is floated to the site, it must be moored and held in position while it is sunk to and into the seafloor. The moorings must resist current and wave forces and must assist in maintaining the caisson stable and in a vertical attitude. This is complicated by the need to build up the caisson walls progressively to give adequate freeboard, which, of course, raises the center of gravity.

Current force can be approximately determined by the following formula:

$$F = CA\rho\frac{V^2}{2g}$$

where C varies from 0.8 for smooth circular caissons to 1.3 for rectangular caissons, A is the area, ρ is the density of water, and V is the average current over the depth of flotation. Steel sheet piles develop high drag, raising the value of C by 20%–30%.

As with all prismatic floating structures, stability requires that a positive metacentric height be maintained. The formula for metacentric height, $\overline{GM}$, is as follows:

$$\overline{GM} = \overline{KB} - \overline{KG} + \overline{BM}$$

where $\overline{KB}$ is the distance from the base to the center of buoyancy, $\overline{KG}$ the distance to the center of gravity, and $\overline{BM} = I/V$.

I is the moment of inertia on the narrowest (most sensitive) axis, whereas V is the displaced volume of water. For typical caissons, a $\overline{GM}$ of +1 m or more should be maintained.

The forces from mooring lines and the frictional forces from any dolphins affect the actual attitude that the structure assumes, often tending to tip it from vertical. When using mooring lines, the lines should be led through fairleads attached near the center of rotation of the structure. However, this location is constantly changing, so the fairlead attachment points may have to be shifted upward from time to time.

Dolphins and "pens" are used on many river caissons, since navigation considerations often preclude mooring lines. These are clusters of piles or small jackets with pin piles and are fitted with vertical rubbing strips on which a caisson slides.

Once the caisson is properly moored on location, it is ballasted down. As it nears the existing river or harbor bottom, the current flow underneath it increases dramatically. When the bottom consists of soft sediments, these may rapidly scour away in the current. To prevent this, a mattress should first be installed.

Fascine mattresses of willow, bamboo, or wood with a filter fabric attached to them are ballasted down with rock. Alternatively, a layer of graded sand and gravel, similar to the combined mix for concrete aggregate, can be placed. The sand on top will scour away, but the final result will be a reverse filter.

To float a prefabricated caisson to the site initially, false bottoms are fitted over the bottom of the dredging wells. Today these false bottoms are made of steel, although timber was used on many of the famous open caissons from the nineteenth and the early part of the twentieth centuries. They are designed to resist hydrostatic pressure plus the additional force of soils during the early phases of penetration. Once the caisson is embedded sufficiently to ensure stability, the false bottoms are progressively removed so that excavation can be carried out through the open wells. This removal is a very critical and dangerous stage, hazardous to both the caisson and the personnel. At this stage, the water level inside should be slightly higher than that outside. Even then, when the false bottom under a particular well is loosened the soil may suddenly surge up, trapping a diver. The caisson, experiencing a sudden release of bearing under one well, may plunge or tip.

Despite many innovative schemes for the remote removal of false bottoms, accidents have occurred. Today's caissons employ a method for gradually reducing the pressure underneath and excavating some of the soil before the false bottom is released and removed. For such constructions, the false bottom is of heavily braced steel, with a tube through it, typically extending to the water surface. The tube is kept full of water and capped, with a relief valve in the cap. After the caisson has penetrated under its own weight and come to a stop, the relief valve is opened, reducing the pressure to the hydrostatic head only. Then the cap is removed. This is done for several (typically, four) wells in a balanced pattern. Then, jets and airlifts may be operated through the tube to remove the soil under these wells. When the caisson has penetrated sufficiently far for safety against tipping, the wells are filled with water; the false bottoms are removed and dredging can be commenced.

6.4.3 Penetration of Soils

Penetration is primarily accomplished by the net deadweight, that is, the total weight of concrete steel and ballast minus the buoyancy. Excavation within the wells is carried down in a balanced pattern until the bearing stratum is reached. Then tremie concrete is placed, which is of sufficient depth to transfer the design bearing pressures to the walls.

The term "cutting edge" is applied to the tips of caisson walls. The external cutting edges are shaped as a wedge, whereas the interior ones may be either double-wedge or square edges. In the past, concern over concentrated local bearing forces led to the practice of making the cutting edges from heavy and expensive fabricated steel. Today, high-strength reinforced concrete is employed, although if obstructions such as boulders, cobbles, or buried logs are anticipated or if the caisson must penetrate rock steel armor should be attached to prevent local spalling.

The upper part of the caisson may be replaced by a temporary cofferdam, allowing the pier shaft dimensions to be reduced through the water column. This reduces the effective driving force on the caisson but maintains and increases its inherent stability.

Penetration requires the progressive failure of the soil in bearing under the cutting edges and in shear along the sides. Frictional shear on the inside walls is reduced by dredging, whereas that on the outside walls is reduced by lubrication, using jets as previously described in Section 6.2.5.

Controlling the penetration is an essentially delicate balancing of these forces, attempting to obtain a slight preponderance of the sinking force. Too great an excess may cause the caisson to plunge and tip or slide sidewise out of position. This is why pumping down water within the caisson, thus reducing buoyancy, is dangerous; it often leads to a sudden inflow of water and soil under one edge, with potentially catastrophic consequences.

Lubricating jets may be operated in groups to limit the total volume of water required at any one time to a practicable pump capacity. In addition to water, bentonite may be injected through the lubricating jets, reducing skin friction. Compressed air may be alternated with water jetting.

Other methods of aiding sinking are used. Vibration may be useful in sinking the caisson through sands, especially when it is accompanied by jetting. This vibration may be imparted by intense vibration of a steel pile located inside the caisson or even by driving on it with an impact hammer to liquefy the sands locally.

Ground anchors inserted through preformed holes in caisson walls may be jacked against the caisson to increase the downward force. They have the advantage that the actual penetration may be readily controlled, regarding both force exerted and displacement.

Since all the parameters of resistance and driving force vary as the caisson penetrates the soil, and because the imbalance is very sensitive to relatively minor changes in these parameters, it is essential to plan the sinking process in closely spaced stages, typically each 2 to 3 m. Values can be precalculated for each such stage, using the values of the soil parameters, changes in contact areas between soil and structure, weights of concrete and steel, and displaced volume. These need not be exact calculations; the soil parameters are estimates only since they are constantly modified by the jetting. However, they are valuable guides for engineering control of operations.

Many warnings have been given by engineers in the past, often based on near failures or actual catastrophes:

1. Verify structural strength during the stages of floating and initial penetration, with consideration for a potentially high resistance under one corner or edge.
2. When removing false bottoms, ensure that the excess pressure underneath has first been relieved.
3. Do not excavate below cutting edges.
4. Check outside soundings continually for evidence of scour, and take corrective steps promptly.
5. Blasting underneath the cutting edges may blow out the caisson walls. Blasting may also cause liquefaction of soils, leading to loss of frictional resistance and sudden plunging. If blasting is needed, do it before starting penetration or, at least, well before the cutting edges reach the hard strata so that a deep cushion of soil remains over the charges.
6. If the caisson tips, avoid drastic corrections. Instead, plan the correction to ensure a gradual return to vertical and to prevent the possibility of tipping over more seriously on the other side. Thus, steps such as digging deeper on the high side and overballasting on the high side should be last resorts and, then, some means mobilized to arrest the rotation as the caisson nears vertical. Jacking against an external dolphin is a safer and more efficient method for correcting tipping (Figure 6.16).

Sinking the caisson should be a continuous process, since once it stops soil friction and shear increase significantly and it may be difficult to restart the caisson's descent.

FIGURE 6.16 Open caisson for Sunshine Bridge across the Mississippi River: this caisson tipped during the removal of false bottoms and is shown being righted by jacking against dolphins.

6.4.4 Founding on Rock

Caissons founded on bare rock present special difficulties. The rock may be leveled by drilling, blasting, and excavation, although blasting introduces the probability of fractures in the underlying rock. Mechanical excavation may therefore be specified for the last meter or two. Rotary drills and underwater road-headers can be used, but the process is long and costly. In some cases, hydraulic rock breakers can be used; in other cases, a hammer grab or star chisel may be used. For the Prince Edward Island Bridge, Canada, soft rock was excavated and leveled by a very heavy clamshell bucket. Hydraulic backhoes and dipper dredges have been used elsewhere. A powerful cutter-head dredge has been planned for use at the Strait of Gibraltar.

6.4.5 Contingencies

Planning should include methods for dealing with contingencies. The resistance of the soil, especially of hard strata, may be greater than anticipated. Obstructions include sunken logs and even sunken buried barges and small vessels, as well as cobbles and boulders. The founding rock or stratum may be irregular, requiring special means of excavating underneath the cutting edges at high spots or filling in with concrete at low spots. One contingency that should always be addressed is what steps to take if the caisson unexpectedly tips.

Several innovative solutions have been used to construct caissons at sites with especially soft sediments. One is a double-walled self-floating concept, without the need for false bottoms. Double-walled caissons of steel were used for the Mackinac Strait Bridge in Michigan. Ballast is progressively filled into the double-wall space, whereas dredging is carried out in open wells.

In the case of extremely soft bottom sediments, the bottom may be initially stabilized by ground improvement, for example, with surcharge by dumped sand, or by stone columns, so that the caisson may initially land on and penetrate stable soil. Great care must, of course, be exercised to maintain control when cutting edges break through to the native soils below, preventing erratic plunging.

This same principle holds true for construction on sand islands, where cutting edges and initial lifts of the caisson may be constructed on a stratum of gravel or other stable material; then, the caisson may be sunk through to the softer strata below. Guides or ground anchors will be of benefit in controlling the sinking operation.

6.5 Pneumatic Caissons

6.5.1 Description

Pneumatic caissons differ from open caissons in that excavation is carried out beneath the base in a chamber under air pressure. The air pressure is sufficient to offset some portion of the ambient hydrostatic head at that depth, thus restricting the inflow of water and soil.

Access through the deck for workers and equipment and for the removal of excavated soil is through an air lock. Personnel working under air pressure have to follow rigid regimes regarding duration and must undergo decompression upon exit. The maximum pressures and time of exposure under which personnel can work are limited by regulations. Many of the piers for historic bridges in the United States, for example, the Brooklyn Bridge, were constructed by this method.

6.5.2 Robotic Excavation

To overcome the problems associated with working under air pressure, the health hazards of "caisson disease," and the high costs involved, robotic cutters (Shiraishi, 1994). have been developed to excavate and remove the soil within the chamber without human intervention. These were recently implemented on the piers of the Rainbow Suspension Bridge in Tokyo, Japan (Figures 6.17 and 6.18).

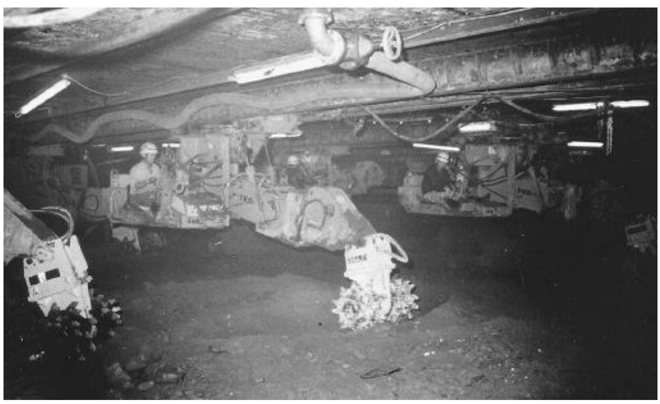

FIGURE 6.17 Excavating within the pressurized working chamber of the pneumatic caisson for the Rainbow Suspension Bridge. (Courtesy of Shiraishi Corporation, Japan.)

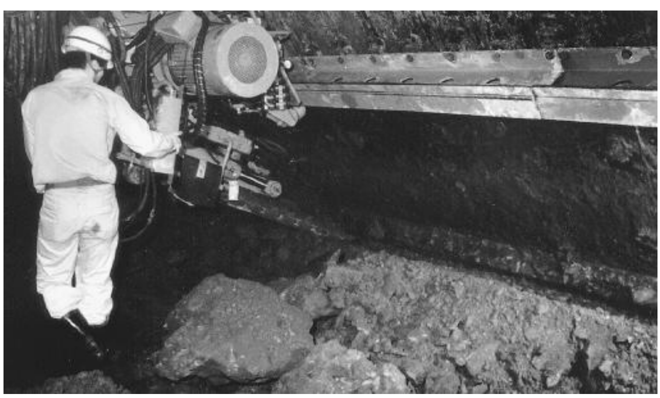

FIGURE 6.18 Excavating beneath the cutting edge of a pneumatic caisson. (Courtesy of Shiraishi Corporation, Japan.)

The advantage of the pneumatic caisson is that it makes it possible to excavate beneath the cutting edges, which is of special value if obstructions are encountered. The great risk is that of a "blowout" in which the air escapes under one edge, which causes a rapid reduction of pressure, followed by an inflow of water and soil, endangering personnel and leading to the sudden tilting of the caisson. Thus, the use of pneumatic caissons is limited to very special circumstances.

6.6 Box Caissons

6.6.1 Description

One of the most important developments of recent years has been the use of box caissons, either floated in or set in place by heavy-lift crane barges (O'Brien et al., 1996). Box caissons, ranging in size from a few hundred tons to many thousands of tons, enable prefabrication at a shore site, followed by transport and installation during favorable weather "windows" and with minimum requirements for overwater labor. Their development has been largely responsible for the rapid completion of many long overwater bridges, cutting the overall time by a factor of as much as three and thus making many of these large projects economically viable.

The box caisson is essentially a structural shell that is placed on a prepared underwater foundation. It is then filled with concrete, placed by the tremie method previously described in Section 6.3.1 for cofferdam seals. Alternatively, sand fill or just ballast water may be used.

Although many box caissons are prismatic in shape, that is, a large rectangular base supporting a smaller rectangular column, others are complex shells such as cones and bells. When a box caisson is seated on a firm foundation, it may be underlain by a meter or two of stone bed, consisting of densified crushed rock or gravel that has been leveled by screeding. After the box has been set, underbase grout is often injected to ensure uniform bearing.

6.6.2 Construction

The box caisson shell is usually the principal structural element, although it may be supplemented by reinforcing steel cages embedded in tremie concrete. This latter system is often used when joining a prefabricated pier shaft on top of a previously set box caisson.

Box caissons may be prefabricated of steel; these were extensively used on the Honshu–Shikoku Bridges in Japan (Figures 6.19 and 6.20). After setting, they were filled with underwater concrete, in earlier cases grout-intruded aggregates but in more recent cases tremie concrete.

For reasons of economy and durability, most box caissons are made of reinforced concrete. Although they are therefore heavier, the concurrent development of very-large-capacity crane barges and equipment has made their use fully practicable. The weight is advantageous in providing stability in high currents and waves.

6.6.3 Prefabrication Concepts

Prefabrication of box caissons may be carried out in a number of interesting ways. The caissons may be constructed on the deck of a large submersible barge. In the case of the two concrete caissons for the Tsing Ma Bridge in Hong Kong, the barge was then moved to a site where it could submerge to launch the caissons. The caissons were floated to the site and ballasted down onto the predredged rock base. After sealing the perimeter of the cutting edge, The caissons were filled with tremie concrete.

In the case of the 66 piers for the Great Belt Western Bridge in Denmark, the box caissons were prefabricated on shore in an assembly-line process (Figures 6.21 and 6.22). They were progressively moved out onto a pier from which they could be lifted off and carried by a very large crane barge to their site. They were then set onto the prepared base. Finally, they were filled with sand and antiscour stone was placed around their base.

FIGURE 6.19 Steel box caisson being positioned for the Akashi Strait Suspension Bridge despite strong currents.

FIGURE 6.20 The Akashi Strait Suspension Bridge is founded on steel box caissons filled with tremie concrete.

FIGURE 6.21 Prefabrication of box caisson piers for the Great Belt Western Bridge.

FIGURE 6.22 Prefabricated concrete box caissons are moved by jacks onto a pier for load-out.

FIGURE 6.23 Large concrete box caissons fabricated in the construction basin for subsequent deployment to site by self-flotation, Great Belt Eastern Bridge.

A similar procedure was followed for the approach piers on the Oresund Bridge between Sweden and Denmark and on the Second Severn Bridge in Bristol, Southwest England. For the Great Belt Eastern Bridge, Denmark, many of the concrete box caissons were prefabricated in a construction basin (Figure 6.23). Others were fabricated on a quay wall.

For the Prince Edward Island Bridge, bell-shaped piers, with an open bottom, weighing up to 8000 t, were similarly prefabricated on land and transported to the load-out pier and onto a barge, using transporters running on pile-supported concrete beams (Figures 6.24 through 6.26). Meanwhile, a shallow trench had been excavated in the rock seafloor, to receive the lower end of the bell. The bell-shaped shell was then lowered into place by the large crane barge. Tremie concrete was placed to fill the peripheral gap between bell and rock.

6.6.4 Installation of Box Caissons by Flotation

Large concrete box caissons have been floated into location, moored, and ballasted down onto prepared bases (Figure 6.27). During this submergence, they are, of course, subject to current, wave, and wind forces. The moorings must be sufficient to control the location; "taut moorings" are therefore used for close positioning.

Taut moorings should be led through fairleads on the sides of the caisson, to permit lateral adjustment of position without causing tilt. In some cases where navigation requirements prevent the use of

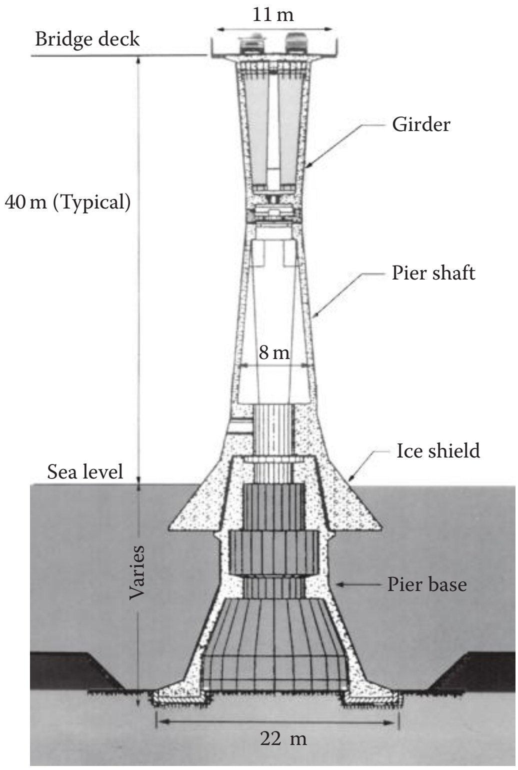

FIGURE 6.24 Schematic transverse cross section at pier location of the substructure for the Prince Edward Island Bridge: note the ice shield, designed to reduce forces from floating ice in the Northumberland Strait.

FIGURE 6.25 Prefabrication of pier bases for the Prince Edward Island Bridge.

taut moorings, dolphins may be used instead. These can be faced with a vertical rubbing strip or master pile. Tolerances must be provided to prevent binding.

Stability is of critical importance for box caissons, which are configured such that the water plane diminishes as they are submerged. It is necessary to calculate the metacentric height, $\overline{GM}$, at every change in horizontal cross section as it crosses the water plane, as described in Section 6.4.2 for open caissons.

FIGURE 6.26 Prefabricated pier shaft and icebreaker for the Prince Edward Island Bridge.

FIGURE 6.27 A prefabricated box caisson is floated into position, the Great Belt Eastern Bridge: note the temporary cofferdam above the concrete caisson.

During landing, as during the similar operation with open caissons, the current under the caisson increases and scour must be considered. Fortunately, in the case of box caissons they are landed either directly on a leveled hard stratum or on a prepared bed of densified stone, for which scour is less likely.

As the base of the caisson approaches contact, the prism of water trapped underneath has to escape. This will typically occur in a random direction. The reaction thrust of the massive water jet will push the caisson to one side. This phenomenon can be minimized by lowering the last meter slowly.

Corrections for the two phenomena of current scour and water-jet thrust are in opposition to one another, since lowering slowly increases the duration of exposure to scour. Thus, it is essential to size and compact the stone of the stone bed properly and also to pick a time of low current, for example, slack tide, for installation.

FIGURE 6.28 Installing precast concrete box caissons for the Second Severn Crossing: an extreme tidal range of 10 m and a high tidal current imposed severe demands on installation procedures and equipment.

FIGURE 6.29 Lifting a box caisson from the quay wall on which it was prefabricated and transporting it to site while being suspended from the crane barge.

6.6.5 Installing Box Caissons by Direct Lift

In recent years, very heavy lift equipment has become available. Jack-up barges, floating crane barges, and catamaran barges have all been utilized (Figures 6.28 through 6.31). Lifts up to 8000 t have been made by crane barges on the Great Belt and Prince Edward Island Bridges.

The box caissons are then set on the prepared bed. Where it is impracticable to screed a stone bed accurately, landing seats may be preset to exact grade under water and the caisson landed on them and tremie concrete filled in underneath.

Heavy segments, such as box caissons, are little affected by current; hence, they can be accurately set to near-exact location in plan. Tolerances on the order of 20–30 mm are attainable.

6.6.6 Positioning

Electronic distance finders, theodolites, lasers, and Global Positioning System are among the devices utilized to control the location and grade of caissons. Seabed and stone bed surveys may be by narrow-beam high-frequency sonar and side-scan sonar. At greater depths, the sonar devices may be incorporated in an remotely operated vehicle (ROV) to get the best definition.

FIGURE 6.30 Setting a prefabricated box caisson on which a temporary cofferdam is mounted, the Great Belt Eastern Bridge.

FIGURE 6.31 Setting a 7000 t prefabricated box caisson, the Great Belt Western Bridge.

6.6.7 Grouting

Grouting or concreting underneath is commonly used to ensure full bearing. It is desirable to use low-strength, low-modulus grout to avoid hard spots. The edges of a caisson have to be sealed by penetrating skirts or flexible curtains, which can be lowered after the caisson is set in place, since otherwise the tremie concrete will escape, especially if there is a current. Heavy canvas or submerged nylon, weighted with anchor chain and tucked into folds, can be secured to the caisson during prefabrication. When the caisson is finally seated, the curtains can be cut loose; they will restrain concrete or grout at low flow pressures. Backfill of stone around the edges can also be used to retain the concrete or grout.

Heat of hydration is also of concern, so the mix should not contain excessive cement. The offshore industry has developed a number of low-heat, low-modulus thixotropic mixes suitable for this use. Some of them employ seawater, along with cement, fly ash, and foaming agents. BFS cement has also been employed.

Box caissons may be constituted of two or more large segments, set one on top of the other and joined by overlapping reinforcement encased in tremie concrete. The segments often are match-cast to ensure perfect fit.

6.7 Present and Future Trends

6.7.1 Present Practice

There is a strong incentive today to use large prefabricated units, either steel or concrete, that can be rapidly installed with large equipment, involving minimal on-site labor to complete. On-site operations, where required, should be simple and suitable for continuous operation. Filling prefabricated shells with tremie concrete is one such example.

Two of the concepts previously described in Sections 6.2 and 6.6 satisfy these current needs: the first is that a box caisson—a large prefabricated concrete or steel section—can be floated in or lifted into position on a hard seafloor. The second is that large-diameter steel tubular piles can be driven through soft and variable soils to be founded in a competent stratum, either rock or dense soils. These tubular piles are especially suitable for areas of high seismicity, where their flexibility and ductility can be exploited to reduce the acceleration transmitted to the superstructure (Figure 6.32). In very deep water, steel-framed jackets may be employed to support the piles through the water column (Figure 6.33). The box caisson, conversely, is most suitable to resist the impact forces of ship collision. The expanding use of these two concepts is leading to further incremental improvements and adaptations, which will increase their efficiency and economy.

Meanwhile, cofferdams and open caissons will continue to play an important but diminishing role. Conventional steel sheet pile cofferdams are well suited to shallow water with weak sediments, but they involve substantial overwater construction operations.

6.7.2 Deepwater Concepts

The Japanese have had a study group investigating concepts for bridge piers in very deep water and soft soils. One initial concept that has been pursued is that of the circular cofferdam constructed of concrete by the slurry wall process (Gerwick and Durnal, 2012) This was employed on the Kobe anchorage for the Akashi Strait Bridge and on the Kawasaki Island ventilation structure for Trans-Tokyo Bay Tunnels, the latter with a pumped-out head of 80 m, in extremely soft soils in a zone of high seismicity (see Section 6.3.1).

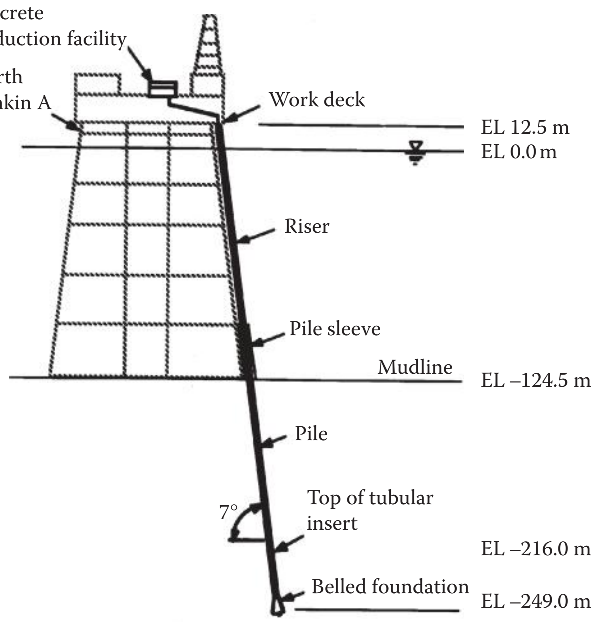

FIGURE 6.32 Conceptual design for a deepwater bridge pier, utilizing prefabricated steel jacket and steel tubular pin piles.

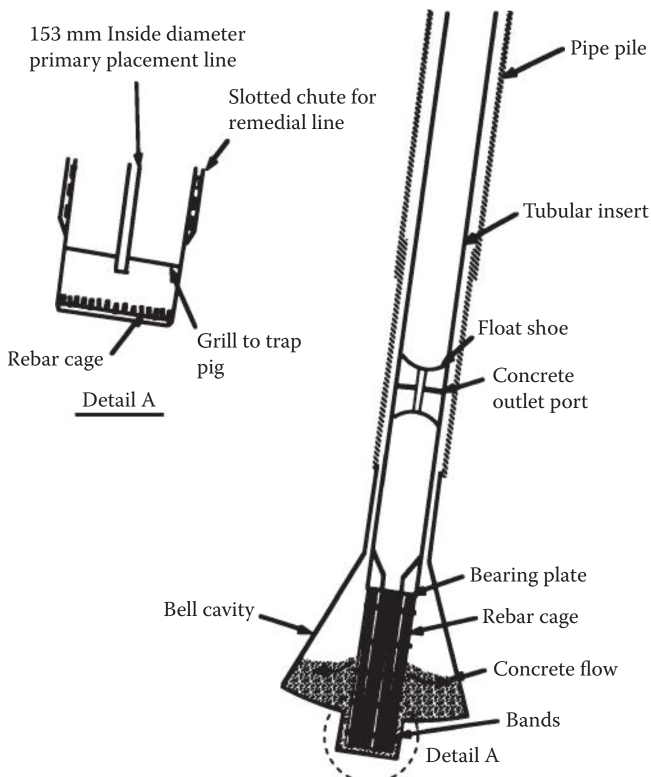

FIGURE 6.33 Belled footing provides greater bearing area for a driven-and-drilled steel tubular pile.

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Floating piers have been proposed for very deep water, some employing semisubmersible and tension "leg-platform" concepts from the offshore industry. Although technically feasible, the entire range of potential adverse loadings, including accidental flooding, ship impact, and long-period swells, needs to be thoroughly considered. Tethered pontoons of prestressed concrete have been successfully used to support a low-level bridge across a fjord in Norway.

Most spectacular of all proposed bridge piers are those designed in preliminary feasibility studies for the crossing of the Strait of Gibraltar. Water depths range from 305 m for a western crossing to 470 m for a shorter eastern crossing. Seafloor soils are highly irregular and consist of relatively weak sandstone, locally known as flysch. Currents are strong and variable. Wave and swell exposure is significant. For these depths, only offshore platform technology seems appropriate.

Both steel jackets with pin piles and concrete offshore structures were investigated. Among the other criteria that proved extremely demanding were potential collision by large crude oil tankers and, below water, by nuclear submarines.

These studies concluded that the concrete offshore platform concept was a reasonable and practicable extension of the current offshore platform technology. Leveling and preparing a suitable foundation is the greatest challenge and requires the integration and extension of present systems of dredging well beyond the current state of the art (Figure 6.34). Conceptual systems for these structures have been developed, which indicate that the planned piers are feasible by employing an extension of the concepts successfully employed for the offshore concrete platforms in the North Sea (Figure 6.35).

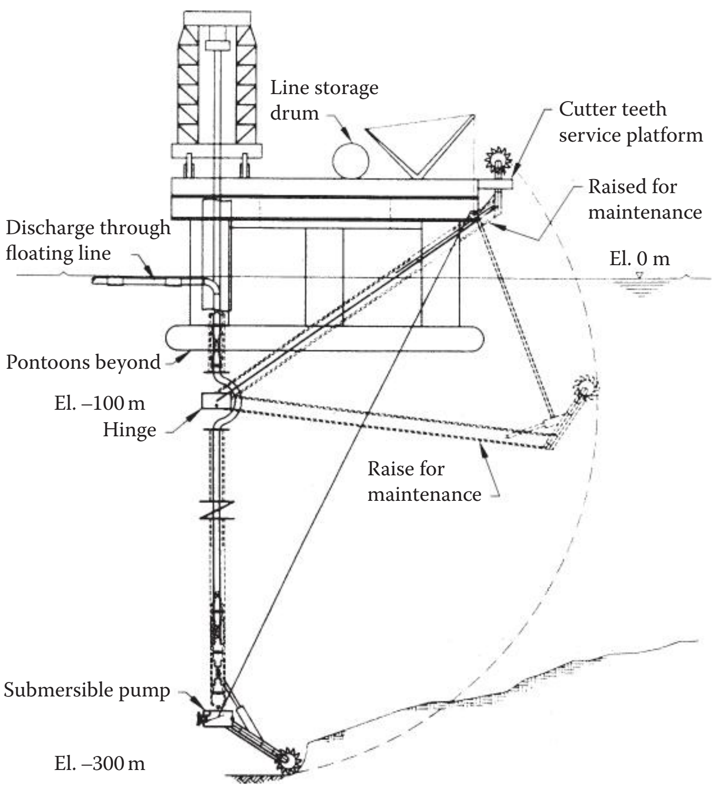
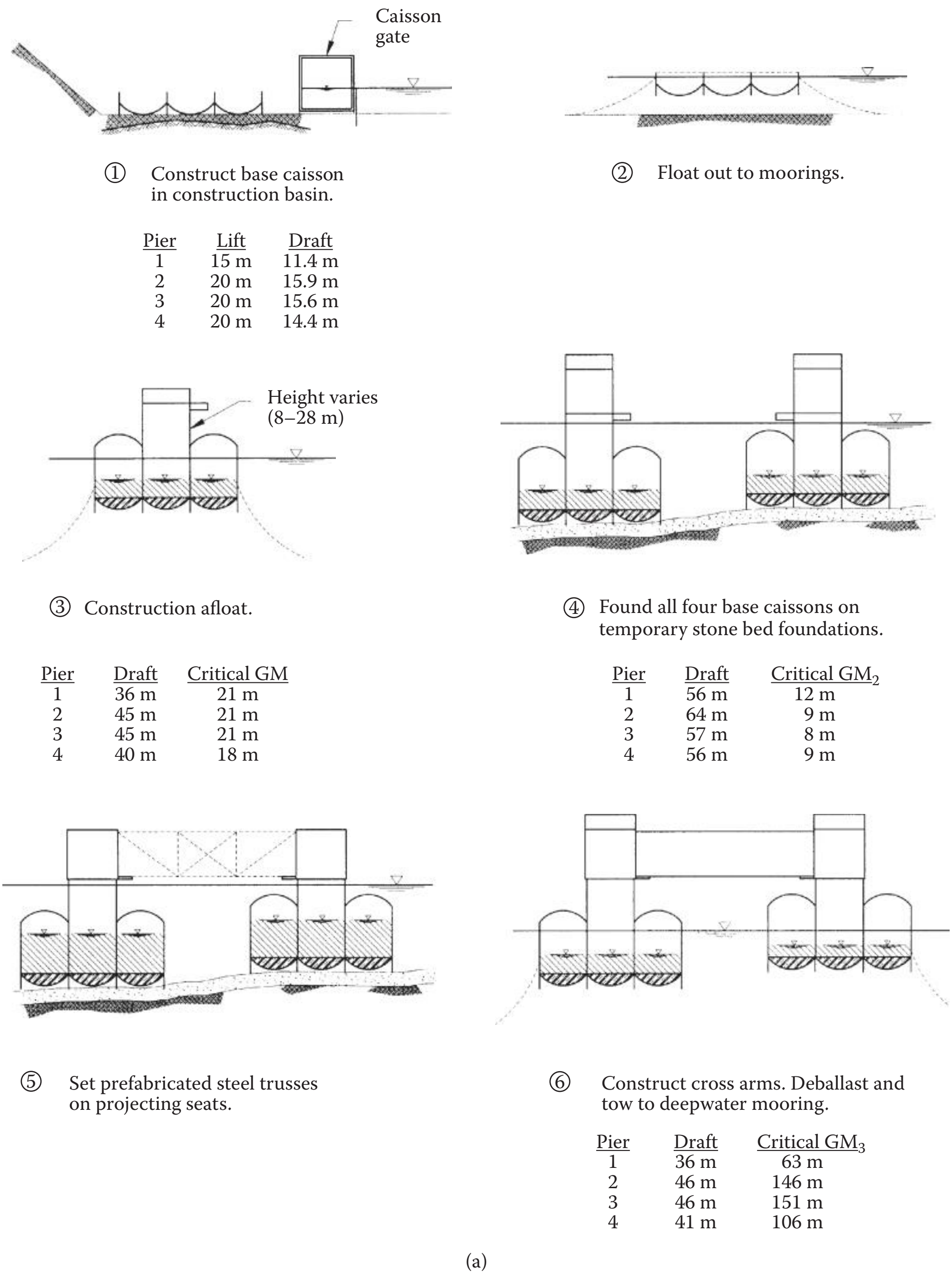


FIGURE 6.34 Concept for the preparation of seabed for seating prefabricated box piers in 300 m water depth, Strait of Gibraltar.



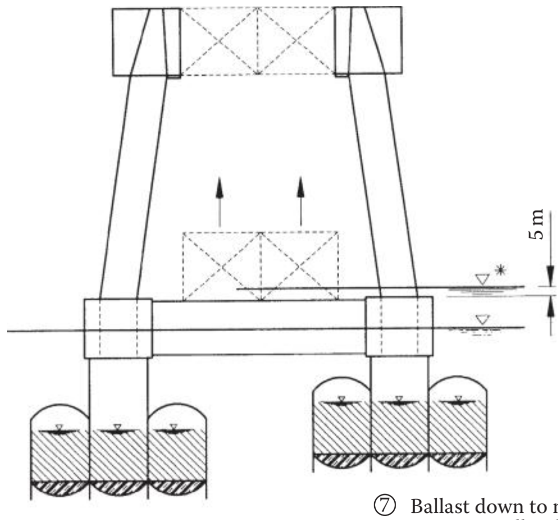
Pier	Lift	Draft
1	15 m	11.4 m
2	20 m	15.9 m
3	20 m	15.6 m
4	20 m	14.4 m

Pier	Draft	Critical GM
1	36 m	21 m
2	45 m	21 m
3	45 m	21 m
4	40 m	18 m

Pier	Draft	Critical GM_2
1	56 m	12 m
2	64 m	9 m
3	57 m	8 m
4	56 m	9 m

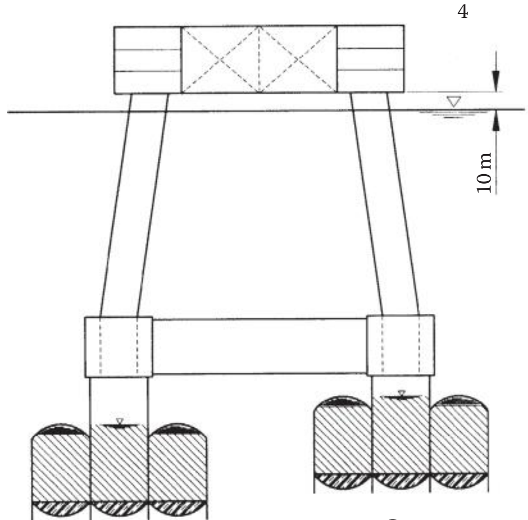
Pier	Draft	Critical GM_3
1	36 m	63 m
2	46 m	146 m
3	46 m	151 m
4	41 m	106 m

FIGURE 6.35 Fabrication and installation concept for piers in 300 m water depth for crossing the Strait of Gibraltar.



⑦ Ballast down to midheight of cross arms. Construct all 4 shafts and conical tops. Raise platform fabrication trusses and secure to shaft top cones upper falsework (Truss weights'- 2000 t).

Pier	Draft*	Critical GM_4
1	96 m	22 m
2	106 m	34 m
3	95 m	36 m
4	97 m	27 m

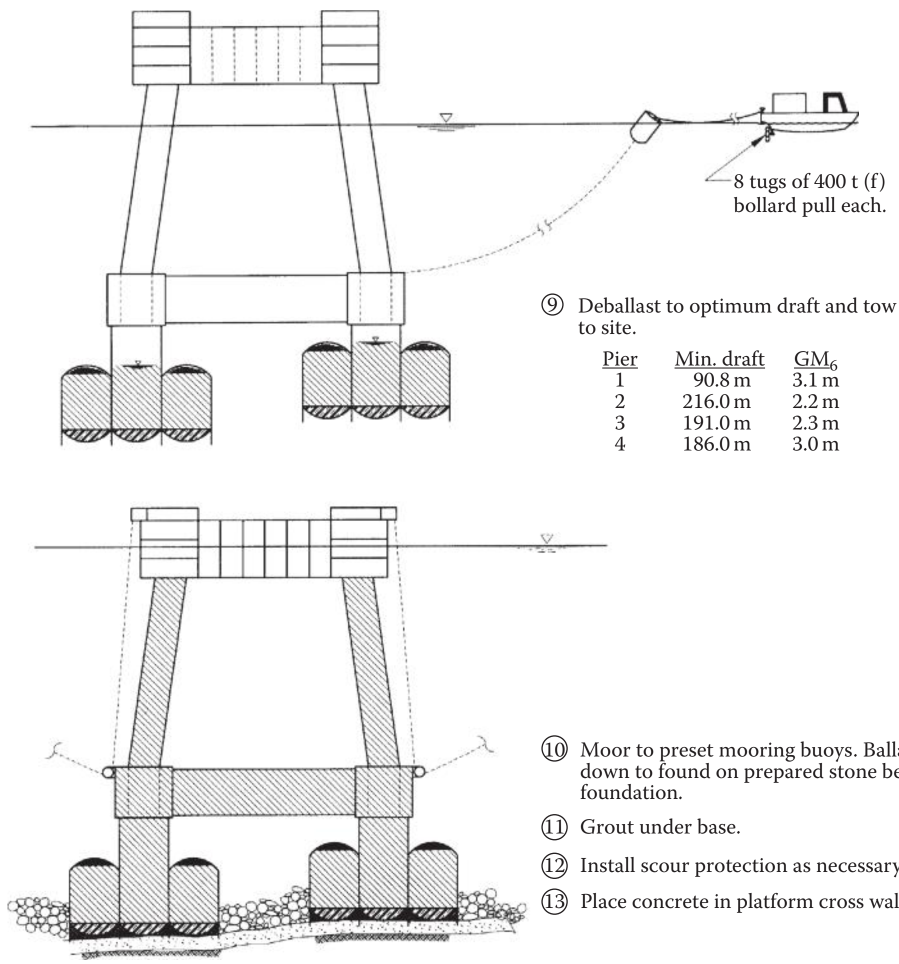


⑧ Ballast down to deep draft. Complete upper platform. Vertical cross walls formed with rebar tied but no concrete placed.

Pier	Draft*	Critical GM_5
1	103 m	5 m
2	281 m	21 m
3	246 m	16 m
4	196 m	6 m

(b)

FIGURE 6.35 (*Continued*) Fabrication and installation concept for piers in 300 m water depth for crossing the Strait of Gibraltar.



(c)

FIGURE 6.35 (*Continued*) Fabrication and installation concept for piers in 300 m water depth for crossing the Strait of Gibraltar.

References

API. 2010. *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms—Working Stress Design,* API RP 2A-WSD (R2010), American Petroleum Institute, Washington, DC.

Gerwick, Ben C. Jr. 1986. *Construction of Offshore Structures*, John Wiley & Sons, New York, NY.

Gerwick, B. C. Jr. and Durnal, P. E. 2012. Cofferdams, in *Handbook of Temporary Structures in Construction*, 3rd Edition, Ratary, W. ed., McGraw-Hill, New York, NY.

O'Brien, J. J., Havers, J. A., and Stubbs, F. W., eds. 1996. *Standard Handbook of Heavy Construction,* 3rd ed., chap. B-11 (Marine Equipment), chapter D-4, (Cofferdams and Caissons), McGraw-Hill, New York, NY.

Shiraishi, S. 1994. Unmanned excavation systems in pneumatic caissons, in Proceedings of *Developments in Geotechnical Engineering*., Balasubramaniam, A. S., Bergado, D. T., Nutalaya, P. and Hong, S. W., eds., A. A. Balkema, Rotterdam, the Netherlands.

7

Accelerated Bridge Construction

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7.1 Introduction: Aging Infrastructure and Changing Needs

Transportation is an important and integral part of the American's overall economy. America's public works have strategically expanded the highway networks in the last century. The building of the Interstate Highway System is one of the best highway infrastructure and largest public works program ever accomplished in the world. When the United States celebrated the 50th Anniversary of the Eisenhower Interstate Highway System in 2006, transportation officials were being reminded how quickly the general bridge infrastructure has been aging and reaching its 50-year design life. The average age of bridges in the America was slightly older than 40 years then. Most pavement design life has a 20-year life expectancy. This explains why the highway networks are always under reconstruction and the standard orange cone becomes a permanent fixture along the highway landscape; there is no end in sight.

With landmark transportation legislations passed by Congress, program funding has been increasing substantially in the past three transportation acts, that is, the Intermodal Surface Transportation Efficiency Act; the Transportation Equity Act for the 21st Century; and the Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users. Replacing aging bridges that carry heavy traffic volumes presents a unique challenge, and traditional method of construction is no longer a viable solution. At the same time, increasing traffic volume and heavy congestion are choking the highway network. During peak travel seasons, as much as 20% of the national highway system is under reconstruction, which further reduces traffic capacity and aggravates congestion (Battles 2004). Congestion impedes the nation's prosperity and impacts mobility. It affects a nation's productivity, mobility, and safety; it increases the costs of doing business. A 2011 Annual Urban Mobility Report by Texas Transportation Institute indicated that the cost of congestion due to extra time and fuel "invoice" in 439 urban areas was $115 billion compared to $113 billion (in constant dollars) in 2006. The wasted fuel amounts to 1.9 billion gallons (Schrank, Lomax, and Eisele 2011).

The American economy strives as a highly mobile society; transportation is about 11% of its gross domestic products. It accounted for 19% of the expenditure of an American household. Travel time reliability is as important to the business communities as it is to consumers. Many businesses and manufacturers plan their productions and services with the just-in-time delivery concept and demand reliability from their shippers. Freight carriers rely on predictable travel times to stay competitive and to maintain good customer service. Americans spend 3.7 billion hours and 2.3 billion gallons of fuel each year in traffic jams (USDOT 2012).

The traveling public is demanding that rehabilitation and replacement be done quickly to reduce congestion and improve safety. As such, conducting "business as usual" in the reconstruction of the highway infrastructure, especially the Interstate System bridges, is no longer acceptable. Public officials are looking beyond the traditional construction methods to provide consistent and predictable level of mobility as well as improving safety. And accelerated bridge construction (ABC) using streamlined engineering processes and prefabricated elements and systems (PBES) demonstrated its worth through several pilot projects and is being accepted as an innovative practice in today's construction environment.

At the time of this writing, there are several research and demonstration projects under development. These projects are considered to be work in progress and new information is being generated continuously. The author captured as much information as possible and will point readers to the organizations that are leading these developments for future follow-up reports. If past can point to how information is being disseminated, the author believes they would continue to be made available through regional and national bridge conferences, webinars, and published trade journals. The Federal Highway Administration (FHWA) and state highway agencies have been posting their latest project information and ABC project–related materials on their own web sites and some of the links are provided at the end of this chapter.

7.2 Motivation for Accelerated Bridge Construction

When the Interstate Highway System network was being built, the routes were mostly new facilities, and managing traffic through the construction zones was not a concern. Majority of those routes today are heavily congested and the cost to maintain traffic control through a work zone can vary anywhere from 10% in rural areas to 40% in urban areas of the total construction cost. In some high-mobility routes and lifeline networks, total closure to traffic flow during reconstruction for long duration is not feasible nor can it be tolerated. Then a facility owner is required to provide temporary detour route and/or bridge, which increases the total project cost. In consideration of the added expenses to constructing and maintaining the detour, the option to get in and deliver the project quickly and get out of the work zone can be a very attractive and cost-effective solution.

Innovative technology and improved process has contributed to making the life cycle costs of ABC bridges more efficient and cost-effective. In recent years, the development of innovative materials provides owners with more durable and high-performance bridges. The construction industry has developed new and heavy lift equipment for lifting and moving much heavier payloads than ever before. These innovations enable a contractor to erect whole or partially completed prefabricated bridges with incredible speed. The conventional concrete bridge construction sequence begins with forming, laying, and tying steel reinforcements, pouring and curing concrete, stripping forms and finishing concrete, and they are in the critical path and control the construction schedule. When one considers the series of sequential work stages in building a bridge, that is, foundation, substructure, superstructure, and parapet, it requires several months to construct even a simple bridge. The use of prefabricated bridge elements and systems (PBES) built off the critical path enables a contractor to erect bridges in a matter of hours as opposed to months. The prefabricated elements or systems fabricated in a shop-controlled environment provide owners with much higher-quality products that last longer and cost less over the life cycle of these bridges.

7.2.1 Objectives of ABC

The main objectives of ABC to accelerated project delivery are the following:

- To reduce the impact on delays to the traveling public
- To produce high-quality and durable products fabricated in shop-controlled environment
- To enhance the work zone safety by minimizing exposure to both the motorists and construction workers by reducing the amount of time it takes to complete a project
- To reduce the overall construction schedule to a fraction of the amount of time normally needed on-site

7.2.2 ABC Bridges Are Covered under the AASHTO LRFD Specifications

Bridges designed under the ABC are governed by the American Association of State Highway and Transportation Officials (AASHTO) Load Resistance Factor Design (LRFD) Specifications (AASHTO 2010a). Bridges erected using ABC techniques and materials used for the bridge elements are governed by the LRFD Construction Specifications (AASHTO 2010b). Roles and responsibilities for bridge owners, designers, contractors, and manufacturers are defined in these specifications.

7.2.3 Prefabricated Bridge Technology Enables Rapid Construction

During a 2004-U.S. sponsored technology scanning tour to Europe and Japan, a group of American engineers and the author have reported on several replacement bridges in urban areas that were erected in a matter of hours and reopened for traffic use after short weekend closures. The key factor in the successful implementation of ABC is the innovative use of PBES with its components fitting together like an erector set. This idea eliminates as much as possible the time-dependent related activities on-site when compared with the conventional construction method. For example, the sequence for constructing a concrete bridge begins with the foundation, then the footing, to be followed by the substructure (pier column and pier caps) and finally the superstructure (girder and deck) with the superimposed loads (parapets, sidewalks, and accessories), being the last elements to complete a bridge. And this is not all of it. For with each element, a contractor has to execute more sequential activities, that is, building the formwork, laying the reinforcing steel, pouring concrete, curing concrete from 7 to 14 days, stripping the formwork, and moving up to the next element or phase of work. Conventional bridge construction is labor-intensive and can go on for months. The use of ABC has been successful and proven repeatedly that a complete bridge can be delivered overnight or over a weekend. This accelerated project delivery schedule is becoming more attractive and desirable for bridge owners who are under constant pressure to "get in and get out" of construction zones, especially in heavily congested and high-mobility highway networks.

7.2.4 Benefits of ABC: PBES

One can easily identify several benefits in the use of ABC concepts. The on-site time required for building a bridge piecemeal and from ground up is greatly reduced, thus reducing overheads for both the owner and contractor. Depending on the specific site conditions, the use of PBES can also minimize traffic disruption, improve work zone safety, and minimize disruption to the environment. The use of PBES improves constructability using heavy lift equipment to quickly erect partial or completed bridge system. The use of PBES also offers higher-quality products because they are manufactured under controlled conditions and brought to the construction site ready for installation. When standard components are to be developed, PBES can lower production costs and will result in lowering the overall and life cycle costs. These sought-after benefits were reported by the 2004 technology scan team members on several projects successfully implemented in Japan and Europe.

ABC can enhance safety through the construction zones for both workers and the traveling public. Each year, the FHWA reported the number of highway deaths caused by traffic crashes within construction work zones and the following is taken from their Work Zone Safety Fact Sheet: "Work zone safety is a growing roadway safety concern. In 2008, there were 720 work zone fatalities; this figure represents 2% of all roadway fatalities for the year. Over four out of every five-work zone fatalities were motorists. In addition, there are over 40,000 injuries in work zones" (FHWA 2011a).

7.2.5 Accelerating Project Delivery Has Become a National Initiative

The concept of accelerating the project delivery was first presented by the Transportation Research Board (TRB) in conjunction with FHWA and the AASHTO, Technology Implementation Group (AASHTO-TIG). Following the completion of two pilot workshops, one in Indiana and one in Pennsylvania, the originating task force, A5T60, passed the concept off to FHWA and TIG to further develop accelerating project delivery. The AASHTO-TIG team did a great job compiling the initial database consisting of design and construction details of projects using PBES. The database formed the basis for the posted information through the FHWA Web site at http://www.fhwa.dot.gov/bridge/.

In the early days of deploying ABC, FHWA undertook a two-prong process and helped several states deploy accelerated construction technology in their projects. The process involved deploying a team of highly experienced technical experts in numerous disciplines and working with the state agency goal owners to scope the project from conception through construction. This process aims at collapsing the time frames in various tasks that affect the project readiness before traffic flow is interrupted for construction staging. The second process focused on collapsing the bridge construction time frame. Once it is identified in the critical path, either PBES or prefabricated modular systems are used to quickly erect the structures.

The speed in which a bridge project can be delivered in a matter of hours is incredible and unheard of until the turn of the twenty-first century. The Utah Department of Transportation (UDOT) has been specifying ABC project delivery schedule as a standard requirement for their bridge replacement projects after they have had great success in accelerating the I-15 corridor reconstruction that required replacing more than 140 bridges in Salt Lake City. Once again, they have delivered an impressive record of over a hundred ABC bridges over the past 4 years. Utah demonstrated that the total construction cost for ABC projects is becoming equal or less than conventional practice. Massachusetts DOT is following suit and advertised 152 construction projects with a combined construction budget valued at \$1.013 billion in 2011 (MADOT 2011).

The use of PBES allows highway agencies to dramatically reduce construction time and traffic-disruptions. The fact that more and more states have deployed and continue to deploy ABC to deliver projects quickly on a national scale is proving that it is a viable and effective solution to the infrastructure renewal. FHWA has been leading the development of connection details, manuals, contract strategies, and guide specifications to advance this innovation. The trade associations representing the concrete and steel industry are also engaged in leading the development of PBES with their own expertise. There is a national trend for bridge owners to quickly deliver construction projects to improve mobility, safety, and customer service in the work zones.

7.2.6 Definition of ABC

The official definition given to ABC has been published by FHWA. Some state DOTs have their own. Here are two definitions that captured some common processes, goals, and elements of ABC:

> The FHWA (2011b) defines ABC as the use of innovative planning, design, materials, and construction methods to specifically reduce the onsite construction time and mobility impacts that occur when building new bridges or replacing and rehabilitating existing bridges.

> The author worked with a team and developed a definition for the Oregon DOT ABC approach as a process that incorporates innovative technologies, contracting methods, design and construction techniques and/or prefabricated elements and systems, to minimize impacts to the traveling public, local community and environment.

This means that any ABC method that utilizes prefabricated bridge elements combines elements into systems or moves a complete bridge span to quickly deliver a project and reopens the highway to traffic is acceptable. The timescale for acceleration can be a small fraction of the conventional construction delivery schedule or it can be as short as a matter of hours or over a weekend.

The FHWA has two time metrics for ABC—on-site construction time and mobility impact time. The on-site construction time is the period of time from when a contractor alters the project site location until all construction-related activity is removed. This includes, but is not limited to, the removal of maintenance of traffic, materials, equipment, and personnel. The mobility impact time is any period of time the traffic flow of the transportation network is reduced due to on-site construction activities. This metric further categorizes into five tiers:

- Tier 1: Traffic impacts within 1–24 hours
- Tier 2: Traffic impacts within 3 days
- Tier 3: Traffic impacts within 2 weeks
- Tier 4: Traffic impacts within 3 months
- Tier 5: Overall project schedule is significantly reduced by months to years

A common reason to use ABC is to reduce the traffic impacts through the construction work zones or to avoid long detours. The traffic flow in the transportation network can be directly impacted by the disruptions caused by on-site construction-related activities. Most of past projects that provided incentives/disincentives for early completion were based on the time delay costs to mobility.

7.3 Decision-Making Frameworks

It is no mystery how a bridge replacement project shows up on the statewide transportation improvement plan or commonly known as STIP. Suffice to say that generally a bridge project is being programmed for replacement based on its structural deficiency or functionally obsolescence, freight mobility, and route priority with input from several stakeholders. When required by an emergency, which is becoming quite common in recent years, due to natural or man-made disasters (e.g., damage caused by earthquake, hurricanes, or accidents), then the process can be somewhat flexible. Once a team has been assigned to develop the project, the best time to decide if and when ABC would be appropriate is at the early project development stage of a construction project.

When deciding if accelerating project delivery is feasible and effective, there should be two parts in the decision process. Part one has to do with fast-tracking the project planning and development stages by having bid documents prepared ahead when constructing a bridge is on the critical path. Part two is to decide how quickly the owner would want to deliver the project using ABC techniques and how soon to open the facility for traffic use. If the scope of work in a given highway project includes both pavement and bridge work, it would not make sense to accelerate the bridge construction when the pavement work would require months later to complete. Further discussions on these two parts are being provided in Sections 7.3.1 and 7.3.2.

7.3.1 Part 1: Decision Consideration Process for Accelerated Construction Technology Transfer

In part one of the decision-making process, the first step is to look at the complete project development process involving other disciplines or topics to determine where the bottleneck is as it relates to the project delivery timeline and what effort should be put in place to collapse the required schedules. The next step is to determine what can be done to streamline the review and permitting process to effectively move the project design forward.

The FHWA posted a manual with guidance recommending the use of a team approach with expert skill sets from several disciplines to review how best to move an ABC forward (FHWA 2005). The author served as a resource member of the FHWA Accelerated Construction Technology Transfer (ACTT) team and provided the bridge expertise service. This is a good approach when complex issues and several disciplines are involved. The project owner or manager should assemble a project team with expertise in each of the required skill set under various disciplines. To determine how quickly a project can be delivered effectively and efficiently, a detail and critical assessment is recommended for each of the following disciplines:

- Right-of-way/utilities/railroad
- Traffic engineering/safety/intelligent transportation systems (ITS)
- Structures
- Innovative financing and contracting
- Worker health and safety
- Geotechnical/materials/accelerated testing
- Long-life pavements/maintenance
- Construction (techniques, automation, and constructability)
- Environment/contest-sensitive design
- Roadway design/geometries
- Public relations

7.3.1.1 Right-of-Way/Utilities/Railroad

If the right of way has yet to be acquired and if there are controversies related to the property that could delay the project schedule, they should resolve them in the project development phase. Likewise, if the utilities must be relocated, advance work must be coordinated with the utility owners so they do not interfere with the project's critical path. If the railroad tracks are within the project scope, advance coordination with railroad officials is highly recommended. They are adamant about maintaining their freight schedules and keeping their tracks open for service at all times.

Right-of-way, utility, and railroad delays seriously impact accelerated operations. More innovative solutions are required for both short- and long-term time-sensitive construction projects. Right-of-way considerations include State laws and procedures covering acquisition and relocation, numbers and types of businesses and residences that may be impacted, readily availability of additional right-of-way, and sometimes, the number of outdoor advertising structures in the project area. Other items to consider are industry responsiveness, incentive-based utility agreements, corridor approaches to utility agreements, contracting for utility work, and nondestructive methods of utility relocation. When applicable, close railroad coordination is essential for a project for construction access or work impacting the railroad lines.

7.3.1.2 Traffic Engineering/Safety/Intelligent Transportation Systems

Enhanced safety and improved traffic management by corridor contracting should be considered. Developing and evaluating contract models may illustrate the best use of incentives to enhance safety and improve traffic flow during and after construction. Evaluating both the construction and maintenance work may help assess traffic and safety issues more fully than the conventional project-by-project approach. Early assessment can result in better information to the traveling public and politicians on the relationships among crashes, delays, mobility, total traffic volume, truck traffic volumes, and the need for lane closures during construction. Implement integrated ITS to communicate construction information to motorists through radio, Internet, and wireless alerts, along with incident management systems/services.

7.3.1.3 Structures (Bridges, Retaining Walls, Culverts, and Miscellaneous)

Accelerating the construction of structures will require deviation from standard practices for design and construction and include early coordination between designers and contractors. A systems approach from the "ground up" will be necessary instead of emphasis on individual components. Prefabrication, preassembly, incremental launching, lift-in, roll-in, and so on, are systems or concepts with proven contributions to accelerating construction and should be understood and receive priority consideration. Designers have several options in structure types and materials to meet design requirements, but identifying the most accommodating system while minimizing adverse project impacts should be the objective. See Section 7.5 for more discussions on construction techniques.

7.3.1.4 Innovative Financing/Innovative Contracting

Align financing options with the goals of the project by matching anticipated cash flow with project management, while recognizing competing priorities for existing resources. Financing tools could include cost-sharing strategies, tolling mechanisms, contractor financing, leveraging techniques, credit assistance, and cost management and containment concepts. Explore state-of-the art contracting practices and obtain a better knowledge of how these techniques could be selected, organized, and assembled to match the specific situations needed on this project. Techniques to be considered include performance-related specifications, warranties, design-build (DB), maintain, operate, cost plus time, partnering escalation agreements, lane rental, incentive/disincentives, value engineering (VE), and any other innovative contracting techniques that would apply to the project. See Section 7.4 for more discussions on contracting strategies.

7.3.1.5 Worker Health and Safety

When handling hazardous materials or toxic substances, it is advisable to bring in the experts who are familiar with regulations and worker's health and safety. This will ensure proper permits and manifests are put in place. Workers are to be monitored with blood tests and physical examinations preconstruction and postconstruction. Depending on the duration of the job, monitoring of worker's health might be needed during the construction.

7.3.1.6 Geotechnical/Materials/Accelerated Testing

Subsurface conditions and issues should be explored to assess their impacts on the project. Based on the topography and geography where a project is located, subsurface investigation may be complicated by traffic volume, environmental hazards, utilities, railroad property, and right-of-way. Pursue options to expedite and facilitate turnaround times in material testing for material acceptance and contractor payment. The use of innovative materials should be explored and encouraged on projects to maximize the creative characteristics of the designer and contractor. By identifying project performance goals and objective, the designer and contractor have the maximum freedom to determine the appropriate methodology for constructing the project.

7.3.1.7 Long-Life Pavements/Maintenance

Bridge work normally includes pavement approaches on either side and should be addressed as part of ABC. It is feasible to acquire pavement designs approaching 50–60 years by specifying to the contractor what is being wanted, rather than how to build the pavement. By identifying and communicating the pavement performance goals and objectives for the pavement, the designer and contractor have the maximum freedom to determine the appropriate methodology. Explore the future maintenance issues on the project including winter services, traffic operations, preventative maintenance, and any other concerns that may impact the operation of the project features.

7.3.1.8 Construction (Techniques, Automation, and Constructability)

Accelerated construction may press the contractor to deliver a quality product in confined time frames and areas, while maintaining traffic. Completion milestones and maintenance and protection of traffic are key elements visible to the traveling public. Allowing contractors to have input on design elements that would impact time or quality during construction can improve the effectiveness and efficiency of the overall project completion. The use of automation to enhance construction equipment performance, construction engineering and surveying, data collection and documentation, and contract administration should be explored and implemented.

7.3.1.9 Environment/Context-Sensitive Design

Scope-of-work and construction activities need to reflect environmental concerns to ensure the most accommodating and cost-effective product while minimizing natural and socioeconomic impacts. Context-sensitive design allows early coordination and conversation with various stakeholders and interest group to incorporate concerns and resolutions.

7.3.1.10 Roadway/Geometric Design

Highway geometrics can greatly impact project funds and integrity. Although designers may have several options meeting design standard requirements, identifying the most accommodating product while minimizing adverse impacts should be the objective.

7.3.1.11 Public Relations

The vast majority of our nation's highway projects involve reconstruction of existing facilities under or adjacent to traffic. It is imperative to partner with local entities and effectively inform the communities and the traveling public to minimize construction delays as well as adverse socioeconomic impacts. Utah's first experience in 2007 with ABC has been such a big success with the public that thousands of spectators showed up to watch the first of their weekend bridge erection. The ABC event was captured and made the evening news on several local and national broadcasts. The public support was tremendous and the UDOT garnished stronger support from their state legislation.

7.3.2 Part 2: ABC Decision-Making Framework

The second part of the decision-making process is when the bridge construction is on the critical path, the project team would decide which of the most effective method of construction should be used to deliver the project. Initially, ABC was largely driven by the industry at a time when owners promoting DB contracting method began to delegate more process and product controls to the contractors.

Contractors are innovative entrepreneurs and they will capitalize on their strengths, experience, and knowledge when given an option.

Bridge owners should be leading the implementation of ABC since this is a change from the traditional project delivery process. There is sufficient information on ABC currently available for any owner to make a good business decision. Although an owner can continue to specify accelerated project delivery requiring short window time, it would benefit the owner to prescribe the use of an ABC method deemed suitable with an option for contractor to offer their own innovation. By prescribing ABC, an owner demonstrates that the prescribed accelerated method of construction is well thought out and it can be done if a contractor chooses to follow the owner's plans. The owner should understand how the behavior of the structure and its components are being affected by the method of construction to ensure their long-term structural capacity and performance. By specifying the construction method, the owner has some control in limiting the stresses in the bridge and its components.

Contractors must make a profit to stay in business, and owners want contractors to stay in business to compete on future bridge projects. Accelerating bridge projects allows contractors to complete more projects in a given amount of time, which should increase the contractor's profits. Bridge owners must share the risks inherent with innovation and help the contractors to be successful on these projects.

Careful planning, design, and implementation are required to realize the significant advantages of prefabricated bridge construction. Decision makers must consider if the job should be fast-tracked, the applicability of the design, the abilities of contractors and suppliers in the target market, access to the project site, and how the construction requirements affect cost and schedule. Other important factors for success of an accelerated bridge project include the owner's and contractor's commitment to see the job through; willingness to share responsibility, control, and risk; and understanding that time is money for all stakeholders. Owners should be able to expect reasonable cost, durable, and fast construction, allowing them to get more projects within available budgets, whereas contractors should be able to make a reasonable profit and have more bidding opportunities.

The author had managed a research project to develop a framework for the objective consideration of the above-mentioned issues when he was employed by the FHWA. The framework is a qualitative decision-making tool to help engineers answer the ultimate question of whether a prefabricated bridge is achievable and effective for a specific bridge location. The framework can be used at various levels of detail in the decision-making process. There are three formats published in this report—a flowchart, a matrix, and a narrative discussion of the various considerations in the decision process. The flowchart (Figure 7.1) serves to guide a high-level assessment of whether a prefabricated bridge is an economical and effective choice for the specific bridge under consideration. The matrix (Figure 7.2) provides the users with a series of Yes–No and Maybe statements that has more elaborations than the flowchart. The narrative section consists of considerations in various categories corresponding to those in the flowchart and matrix, with discussions and references for use in making a more in-depth evaluation on the use of prefabrication. The flowchart, matrix, and considerations section may be used independently or in combination, depending on the user's desired depth of evaluation. The FHWA (2006) published this decision-making framework and is available free of charge on their web site at http://www.fhwa.dot.gov/bridge/prefab/pubs.cfm.

The author is involved with a newly developed software tool known as ABC Analytic Hierarchy Process (AHP) Decision Tool, and it allows a decision maker and the project team to provide quantitative values or weighted criteria to determine the utility values of different alternatives. More discussion of this tool is provided in Section 7.3.4.

7.3.3 UDOT (Utah) Decision Making for ABC Projects

UDOT implements ABC as a standard practice for project delivery, efficiency, and fast-track construction. From their initial success in delivering ABC projects, UDOT adopted some lofty goals or themes for all the project decision in Utah. These themes capitalize on accelerating project delivery, minimizing

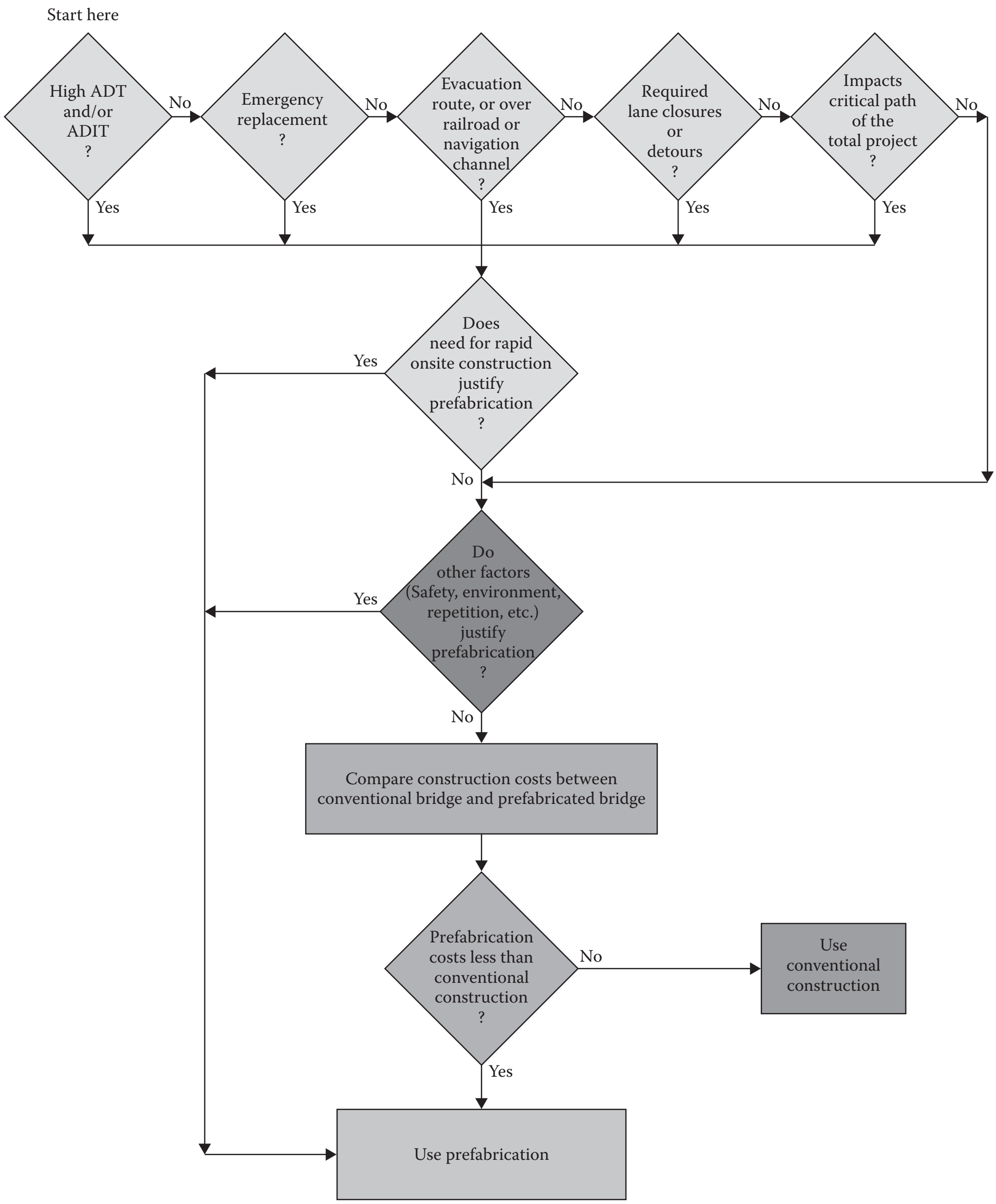

FIGURE 7.1 Flowchart for high-level decision making. (From FHWA, *Decision-Making Framework for Prefabricated Bridge Elements and Systems (PBES), FHWA-HIF-06-030*, Federal Highway Administration, Washington, DC, 2006.)

maintenance of traffic costs during construction, encouraging innovation, and getting a good price. ABC has proven to be successful to UDOT in supporting these themes. UDOT developed an ABC rating approach to determine its direction on the use of ABC for their projects. This process involves measured responses to each factor and allows input to be weighted for the factors that deliver according to the Department's ABC themes. The weighted factors are used to calculate an ABC rating; a high rating means

Question	Yes	Maybe	No
Does the bridge have high average daily traffic (ADT) or average daily truck traffic (ADTT), or is it over an existing high-traffic-volume highway?			
Is this project an emergency bridge replacement?			
Is the bridge on an emergency evacuation route or over a railroad or navigable waterway?			
Will the bridge construction impact traffic in terms of requiring lane closures or detours?			
Will the bridge construction impact the critical path of the total project?			
Can the bridge be closed during off-peak traffic periods, e.g., nights and weekends?			
Is rapid recovery from natural/manmade hazards or rapid completion of future planned repair/replacement needed for this bridge?			
Is the bridge location subject to construction time restrictions due to adverse economic impact?			
Does the local weather limit the time of year when cast-in-place construction is practical?			
Do worker safety concerns at the site limit conventional methods, e.g., adjacent power lines or over water?			
Is the site in an environmentally sensitive area requiring minimum disruption (e.g., wetlands, air quality, and noise)?			
Are there natural or endangered species at the bridge site that necessitate short construction time windows or suspension of work for a significant time period, e.g., fish passage or peregrine falcon nesting?			
If the bridge is on or eligible for the National Register of Historic Places, is prefabrication feasible for replacement/rehabilitation per the Memorandum of Agreement?			
Can this bridge be designed with multiple similar spans?			
Does the location of the bridge site create problems for delivery of ready-mix concrete?			
Will the traffic control plan change significantly through the course of the project due to development, local expansion, or other projects in the area?			
Are delay-related user costs a concern to the agency?			
Can innovative contracting strategies to achieve accelerated construction be included in the contract documents?			
Can the owner agency provide the necessary staffing to effectively administer the project?			
Can the bridge be grouped with other bridges for economy of scale?			
Will the design be used on a broader scale in a geographic area?			
Totals:			

Note: One or two of the above factors may warrant the use of prefabrication to achieve rapid and limited-impact onsite construction. Alternatively, the user may wish to assign weights to the above questions based on the unique circumstances of the project in order to determine whether prefabrication should be used. In any case, prefabrication offers advantages for projects with a majority of "Yes" responses; a more detailed evaluation using the considerations in the next section would then be appropriate.

FIGURE 7.2 Matrix questions for high-level decision. (*From FHWA, Decision-Making Framework for Prefabricated Bridege and Systems (PBES), FHWA-HIF-06-030*, Federal Highway Administration, Washington, DC, 2006.)

ABC would have a high probability of fulfilling the Department's themes and therefore would be the preferred method of project delivery. The ABC rating excel spreadsheet is available on UDOT Web site.

UDOT identified eight measures of project constraints to the ABC decision process. They are average daily traffic, delay/detour time, bridge classification, user costs, economies of scale, applicable typical details, safety, environmental issues, and railroad impacts. These are well-established criteria and they are supported by most highway project decision makers. Under these project constraints, ABC has beaten conventional construction method hands down.

The author recommends the reader to check out the UDOT ABC Web site for more information on the ABC rating and decision flow chart at http://www.udot.utah.gov/.

7.3.4 ABC: Analytic Hierarchy Process Decision Tool

In decision-making process involving alternatives weighted against multiple criteria, it is difficult to mentally keep track of which criterion or how many criteria would end up driving a decision. This accumulative scoring can be done using public domain computer software developed under a pooled fund study as given in the following discussion. In the project planning and design stages, a decision-maker would be working with general assumptions and rough cost estimates based on previous project experience of similar background and lessons learned. More often than not, the available information previously gathered is adequate for an experienced engineer to assess what is more or less important in the project. The AHP is a method that calculates subjective assessments of relative importance to a set of overall scores or weights in each of the pairwise comparison between the criteria.

The author led a FHWA-sponsored Transportation Pooled Fund Study, TPF No. 221(5), with several states (CA, IA, MN, MT, OR, TX, UT, WA, and FHWA) and guided the development of a decision analysis program using the AHP. This program was developed by Toni Doolen (2011) using Microsoft Visual Studio.NET as a stand-alone application. The tool allows the project team to analyze the various applicable and weighted criteria in a pairwise comparison to determine whether ABC is preferred over conventional construction. With the input provided by either a designer or a project team, it would capture the preferred alternative based on the controlling criteria and compute utility value for each criterion. The AHP Program and manual is posted on the FHWA Web and is available to the public for free from http://www.fhwa.dot.gov/bridge/abc/fast.cfm. The AHP program should be loaded onto a personal desktop or laptop computer with the "dotNetFx40_Full_x86_x64.exe" installed for it to run.

This program may be used with the input provided by the bridge engineer alone if he or she has all the available information and feels comfortable assessing the relative importance between any two given criteria. When a project is complex and involves issues or concerns by other disciplines, it would be appropriate for the project team to provide the input and thus build consensus in their decision-making process. The input can be collected with a survey form or entered directly into the program data fields. The input may be changed when more information becomes available to better gauge the relative importance between any given paired criteria or subcriteria. The output produces five files and provides a quantitative analysis yielding three pieces of information to assist in the decision making. One analysis will determine the utility value between two alternates, that is, conventional and ABC. The second piece is the criteria utility contribution and the third is the synthesized criteria weights. More discussion and explanation will be provided in the following example.

7.3.4.1 Established Criteria and Subcriteria for ABC Decision

Generally speaking, most transportation project decision-making processes require some criteria that are important and specific to each site. Figure 7.3 depicts five main level criteria that were established and they represent the common ones used by several states for decision with ABC projects. Within each main level, criterion is further defined by a subcriterion that expands to differentiate its elements. The definitions that were established by the members in the pooled fund study for each criterion are provided in Table 7.1.

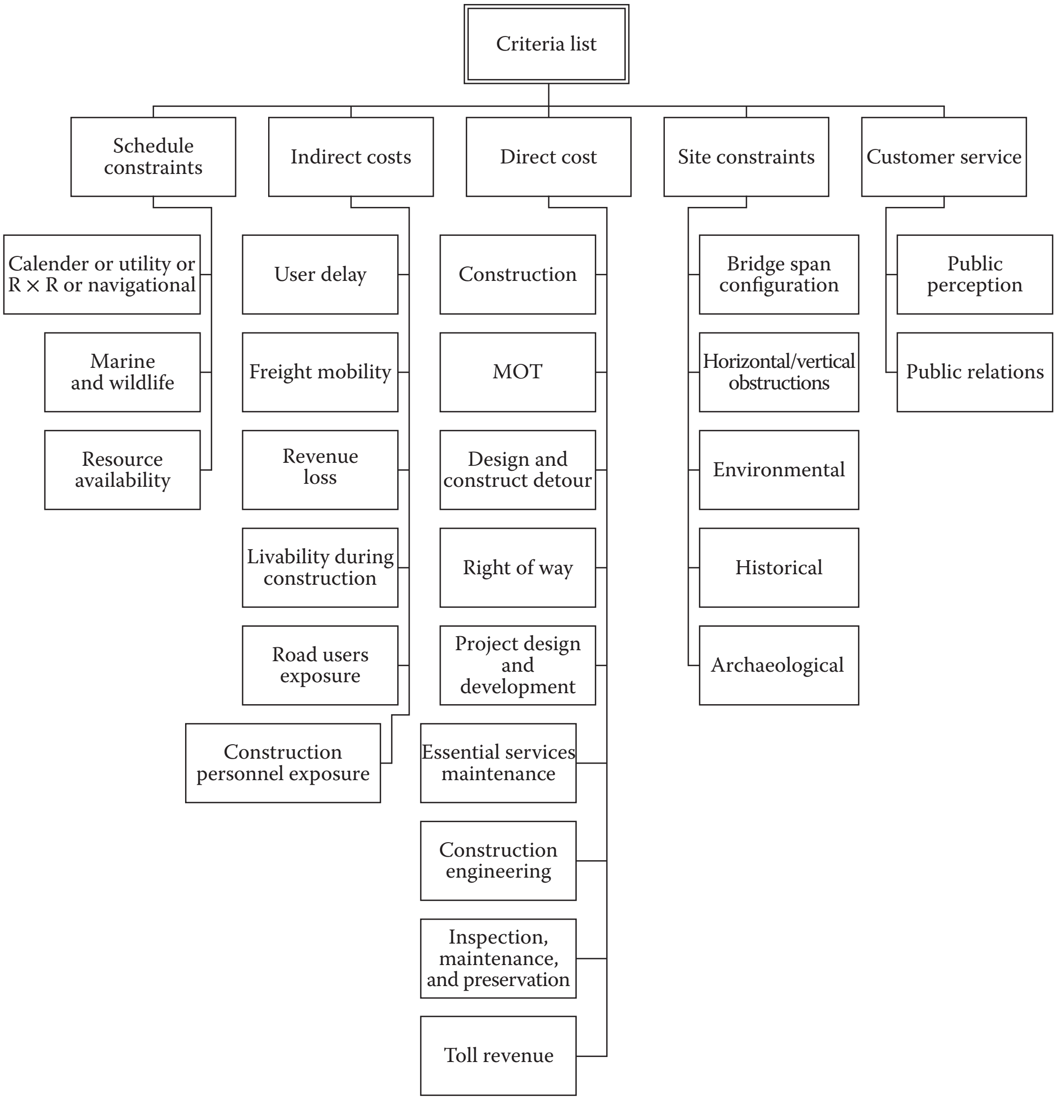

FIGURE 7.3 Main and subcriteria for ABC decision. (From Doolen, T., *Accelerated Bridge Construction (ABC) Decision Making and Economical Tool, TPF-5(221)*, FHWA Transportation Pooled Fund Program, http://www.pooledfund.org/Details/Study/449, 2011.)

7.3.4.2 Demonstration of the AHP Tool in Two Bridge Project Examples

Example 1 is a typical small local agency bridge located outside the edge of a small town using the AHP analysis. The results in this example favor conventional method of construction as noted in Figure 7.4, which depicts the synthesized criteria weights, and the contribution by each individual criterion is as follows:

1. Direct costs: 42.2%
2. Schedule constraints: 26.5%
3. Site constraints: 20.3%
4. Customer service: 7.6%
5. Indirect cost: 3.4%

TABLE 7.1 Definition List of Main Criteria and Subcriteria

High-Level Criteria	Subcriteria	Definition
Direct costs	Construction	This factor captures the estimated costs associated with the construction of the permanent structure(s) and roadway. This factor includes premiums associated with new technologies or innovative construction methods. Premiums might result from factors such as contractor availability, material availability, and contractor risk. It may include incentive/bonus payments for early completion and other innovative contracting methods
	Maintenance of traffic (MOT)	This factor captures the MOT costs at the project site. MOT costs may impact preference due to its impact on total costs. This factor includes all costs associated with the maintenance of detours before, during, and after construction. Examples of this factor include installation of traffic control devices and maintenance of detour during construction including flagging, shifting of traffic control devices during staged construction, and restoration associated with the temporary detours upon completion of construction
	Design and construct detours	This factor captures the costs to design and construct temporary structures and roadways to accommodate traffic through the project site
	Right of way (ROW)	This factor captures the cost to procure ROW. This factor includes either permanent or temporary procurements/ easements
	Project design and development	This factor captures the costs associated with the design of permanent bridge(s) and costs related to project development based on the construction method
	Maintenance of essential services	This factor captures the costs associated with the need to provide essential services that may be impacted by the construction selected. Examples of this factor include alternate routes or modes of transportation to provide defense, evacuation, emergency access to hospitals, schools, fire station, law enforcement, and so on. This criterion is for situations where measures are needed to be implemented beyond those already considered in the "MOT" and "design and construct detours" criteria
	Construction engineering	This factor captures the costs associated with the owner's contract administration of the project
	Inspection, maintenance, and preservation	This factor captures the life cycle costs associated with the inspection, maintenance, and preservation of individual bridge elements
	Toll revenue	This factor captures the loss of revenue due to the closure of a toll facility
Indirect costs	User delay	This factor captures costs of user delay at a project site due to reduced speeds and/or off-site detour routes
	Freight mobility	This factor captures costs of freight delay at a project site due to reduced speeds and/or off-site detour routes
	Revenue loss	This factor captures lost revenues due to limited access to local business resulting from limited or more difficult access stemming from the construction activity
	Livability during construction	This factor captures the impact to the communities resulting from construction activities. Examples include noise, air quality, and limited access
	Road users exposure	This factor captures the safety risks associated with user exposure to the construction zone

(*Continued*)

TABLE 7.1 (*Continued*) Definition List of Main Criteria and Subcriteria

High-Level Criteria	Subcriteria	Definition
	Construction personnel exposure	This factor captures the safety risks associated with worker exposure to construction zone
Schedule constraints	Calendar or utility or railroad (RR) or navigational	This factor captures the constraints placed on the project that might affect the timing of construction as a result of weather windows, significant or special events, RR, or navigational channels
	Marine and wildlife	This factor captures the constraints placed on the project by resource agencies to comply with marine or wildlife regulations. Examples include in-water work windows, migratory windows, and nesting requirements
	Resource availability	This factor captures resource constraints associated with the availability of staff to design and oversee construction. For example, a state may be required to outsource a project, which may result in additional time requirements
Site constraints	Bridge span configurations	This factor captures constraints related to bridge span configurations. This element may impact owner preference regarding bridge layout, structure type, or aesthetics
	Horizontal/vertical obstructions	This factor captures physical constraints that may impact construction alternatives. Examples include bridges next to fixed objects such as tunnels, ROW limitations, sharp curves or steep grades, or other urban area structures that constrain methods and/or bridge locations
	Environmental	This factor captures the constraints placed on the project by resource agencies to minimize construction impacts on natural resources including marine, wildlife, and flora
	Historical	This factor captures historical constraints existing on a project site
	Archaeological constraints	This factor captures archaeological constraints existing on a project site
Customer service	Public perception	This factor captures both the public's opinion regarding the construction progress and their overall level of satisfaction
	Public relations	This factor captures the costs associated with the communication and management of public relations before and during construction

Source: Adapted from Doolen, T., *Accelerated Bridge Construction (ABC) Decision Making and Economical Tool, TPF-5(221)*, FHWA Transportation Pooled Fund Program, http://www.pooledfund.org/Details/Study/449, 2011.

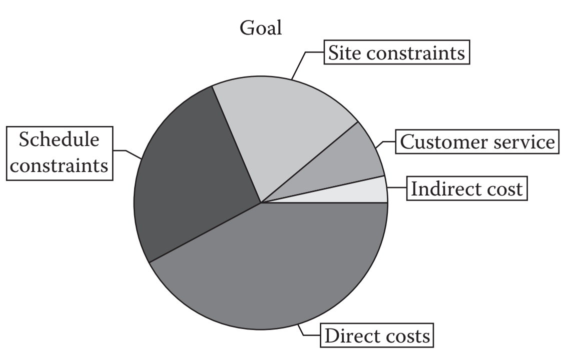

FIGURE 7.4 Synthesized criteria weights for project example 1.

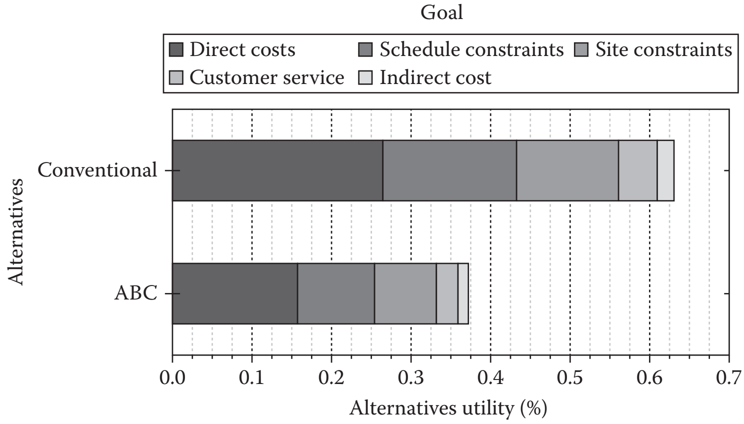

FIGURE 7.5 Project results for example 1 using the Analytic Hierarchy Process software.

This means the assessed project carried a 42.2% weight when direct costs are considered. The direct costs are defined in Table 7.1. For this simple span bridge, it mainly consisted of the estimated costs associated with the construction of the permanent structure and road, maintenance of traffic at the project site, right-of-way acquisition for the permanent or temporary easements for building adjacent to the existing bridge, and the engineering costs. There is a short detour of less than 2 miles to handle local traffic, and the schedule and site constraints contributed 26.5% and 20.3%, respectively. The least issues of concern are the customer service (7.6%) and indirect coast (3.4%) since the detour is provided and there are no further expenses required to address user delays, freight mobility, and jobsite safety.

The bar charts in Figure 7.5 showed the result of the alternative utility favoring conventional construction over ABC by a ratio of 1.7:1 (0.629/0.371). This chart also showed the synthesized criteria weights for the conventional and ABC alternates. The figures for the bar charts are as follows:

1. Direct costs: ABC: 15.7% Conventional: 26.5% Subtotal: 42.2%
2. Schedule constraints: ABC: 9.8% Conventional: 16.7% Subtotal: 26.5%
3. Site constraints: ABC: 7.5% Conventional: 12.8% Subtotal: 20.3%
4. Customer service: ABC: 2.8% Conventional: 4.8% Subtotal: 7.6%
5. Indirect cost: ABC: 1.3% Conventional: 2.1% Subtotal: 3.4%

Example 2 is bridge replacement project on an existing causeway of more than 2 miles long. The bay area situated between two cities is home to oyster colonies and migratory birds. It is a birdwatcher's paradise with busy tourist traffic year-round. The proposed new bridge superstructure is to be consisted of spans ranging from 100- to 150-foot prestressed concrete girders. The substructure is to be consisted of cast-in-place (CIP) caps on trestle piles. Contractors may elect to propose precast bent caps as alternate construction. The AHP decision tool was used to compare the precast concrete (PCC) option versus the CIP option. The required data for this analysis was provided by the project leader. Based on the generated output, the PCC alternative is highly preferable over the CIP alternative for the project (Figure 7.6). The calculated utilities for the PCC and CIP alternatives are 0.720 and 0.280, respectively. This means the utility value for the PCC alternate is 2.57 times in favor over the CIP alternate (Figure 7.7).

Figure 7.7 presents the high-level criteria weights for Example 2 bridge project. The results indicate that "Site Constraints" and "Schedule Constraints" have the greatest impact on the decision to choose PCC as the suitable alternative.

In summary, these two examples using the AHP tool provided documentation on how the decision was made, what criteria were used, and why it made sense for choosing or not choosing an ABC method. These charts can be incorporated into a word document or as part of the project record.

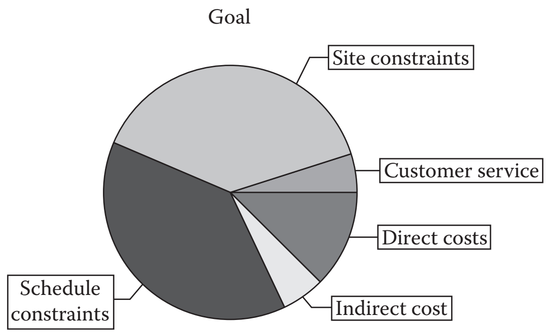

FIGURE 7.6 Synthesized criteria weights for project example 2.

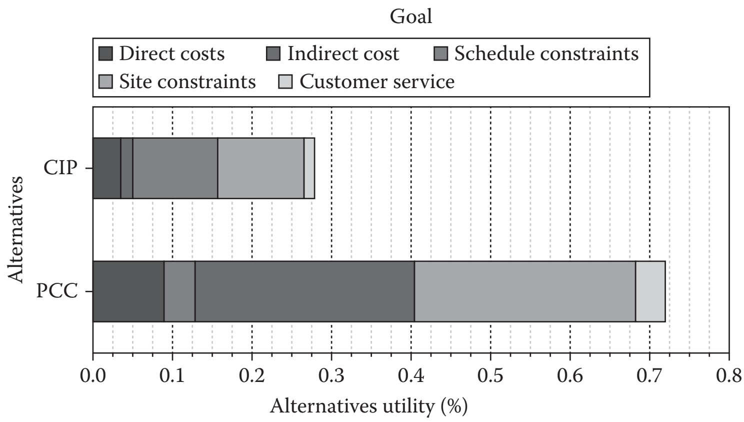

FIGURE 7.7 Project example 2 results using the Analytic Hierarchy Process decision tool.

7.4 Innovative Contracting Methods for Rapid Delivery

The bridge engineering community and construction industry have made great strides in developing innovations to deliver ABC projects since VE was first introduced. The VE provisions allow a contractor to be innovative and to propose alternate solutions without compromising the quality and integrity of the end product. The cost savings realized from a VE change is shared with the owner. Many contractors' innovations were introduced as a result of the VE provisions that have become standard boilerplate language in highway and bridge construction contracts today.

The FHWA has issued innovative contracting guidance under the Special Experimental Projects (SEP-14). Some examples are DB, consultant management-general contractor (CMGC), best value, and cost-plus time variations methods of contracting. A bridge owner should coordinate with his own procurement office to determine what contracting strategies would be most suitable given each state procurement rules and regulations being somewhat different. These provisions will be further discussed in the following as they relate to the contract strategies.

7.4.1 Use of Design-Build Contract

Several alternate contracting approaches other than the conventional design-bid-build are available in contract administration to enable an owner to achieve accelerated project delivery. The common method of most early ABC project delivery was mostly done through a DB contract. The design and project construction are carried out by a single contractor who hired a design consultant firm to perform the design based on how the contractor would build the bridge. The owner established preliminary contract drawings with sufficient details (also known as up to 30% plans) to define the type, size, and location, so the end results are clearly understood by the parties. The contract document specifies the required design standards and minimums. The scope is well defined in the contract and allows the contractor to estimate the job to adequately bid it. The typical project owner specified the limited work window or schedule in which a facility may be closed to traffic for construction. Some limits may contain a maximum allowable number of times and/or durations for a specific time and day when the facility may be closed to traffic or when work could be carried out. If a contractor fails to reopen the facility by the specified time, a disincentive or penalty is assessed against the contractor.

The significant benefit in DB is the potential time savings since both the design and construction activities are under the control of the contractor. It allows the contractor to optimize his work force, equipment, and project scheduling. The construction can begin with the foundation work while the superstructure is being designed. The time it takes to deliver a project is much quicker than the conventional design-bid-build contract.

7.4.2 Use of Incentive and Disincentive or No-Excuse Bonus

The incentive or disincentive clause should specify the amount for accomplishing or not the specified milestones. It is a good strategy to motivate the contractor to do his best and the incentive amount must be adequate to reward the contractor for going the extra mile to accomplish the job. Most states determine the amount of incentive based on the user delays using traffic data and detour length as starting points. The engineering team would estimate what it would take for the shortest and reasonable amount of time needed to complete a project and establish a schedule for project delivery. The reward goes to the contractor when he delivers ahead of schedule. For example, a project is to be scheduled for completion in 30 days and the contractor delivers the work in 12 days; his reward is 18 days of incentive payment. The author has seen some incentive/disincentive amounts that ranged from \$50K to \$250K per day with specified maximum number of days. Successful contractors completed the job ahead of time and collected millions of dollars; no excuse is allowed or given.

7.4.3 Use of Cost-Plus-Time Bidding: A (Contract Bid Items) + B (Time) Method

The Cost-Plus-Time contracting, also referred to as the A + B method, involves the defined time with specified associated cost, in the low-bid determination. Each bidder submits his bid consisting of the contract bid items in the dollar amount for the work to be performed and the number of calendar days estimated by the bidder to complete the job. The bid for award consideration is based on the combination of the formula: (cost in work items A) + (number of days in B × road user cost/day). For each day, the contractor finishes the job early than what he bid, and he will receive an award amount as specified per day. The quicker he can get the job done, the larger is the incentive award. On the contrary, if the contractor lags behind, he will be assessed a penalty or disincentive equal of the amount specified per day. This is a good approach because it allows competition on both the contract items and specified time.

7.4.4 Use of Advance or Phase-In Work

In the use of prefabricated bridge elements or systems or phase-in work, early letting of portions of the project would allow the work to move out of the critical path that impacts the project site schedule. This enables a contractor or supplier to order the materials, schedule the fabrication, and coordinate the special equipment needed to erect the bridge. Some of the heavy-duty cranes or special equipment may take months to be brought in and assembled on-site before they can be put into operation.

7.4.5 CM/GC (Construction Manager/General Contractor)

Under the CM/GC method of procurement, it allows an owner to hire a construction manager to act as a project manager during the design and as a general contractor after the design. This enables the construction manager to coordinate with the project designer to resolve any potential design issues and better manage the identified risks associated with the project. This contracting method provides the owner with greater control over the price, schedule, and scope since the construction manager must provide all this information, his analyses, and recommendation to the owner. When the design package is completed, the construction manager solicits bids from subcontractors and other general contractors and administers the construction phase through completion. This offers more accurate pricing for the actual method of construction being used in the negotiation and to stay within the owner's budget. The main general contractor may be required to self-perform a minimum portion of the work. This method is suitable for large projects with several bridges and allows the contractor to tailor the job to his strengths and deliver ABC project efficiently and effectively.

7.4.6 Bundling Projects

With the aging infrastructure deteriorating rapidly, several bridges along a stretch of the highway corridor may have to be replaced. Instead of doing one project at a time, there is an opportunity to bundle the bridges together under one project by taking advantage of the economies of scale. This allows a contractor to be innovative in amortizing his investment in the formwork and equipment over these projects. For example, the UDOT let out a bundle of seven bridges under a single contract and the contractor came in and set up a production plant prefabricating the bridges. They coined the plant "bridge farm" because the bridges were fabricated in this single location along an interstate highway at proximity to those bridges to be replaced and quickly moved them into their positions during weekend closures.

There are other forms of innovative contracting strategies that take on variations from the above. They all serve to provide an incentive or reward the contractor for accomplishing the owner's goal of delivering faster than conventional practice. The FHWA published guidelines on the use of various innovative contracting methods and more information can be found at http://www.fhwa.dot.gov/programadmin/contracts/050495.cfm.

7.5 Innovative Techniques Used in Rapid Construction

Bridge construction has come a long way from the days of using horse-drawn power to diesel engine–powered cranes and transporters. The American construction industry has evolved in the use of heavy-duty equipment for erecting whole or partially completed bridges over land and bodies of water. When the bridge engineering community talks about erecting bridges with prefabricated systems today, it is talking about "bridge movement" systems. It becomes obvious when bridges are being moved into place by launching longitudinally from one end to the other, skidding sideway, floating with barges, or lifting/jacking into place. These methods are more common today because of the demand for accelerated project delivery, and construction equipment has much improved. There is a new technique in bridge movements in the use of self-propelled modular transporters (SPMT), which is popular in European countries. Flown-in with helicopter is the last category; however, except in remote or hard to reach areas, it is rarely use due to high expenses and difficultly to erect heavy components suspended in the air. A separate and detailed discussion for each method is presented in the following sections.

7.5.1 Bridge Launching

Launching a bridge from one end of the abutment to the other is usually done with hydraulic jacks and roller bearings. There are many variations in the bridge launching technique. For example, the launching could be done with full or partial support from below or above. A fully support launch would require

FIGURE 7.8 Spans were assembled behind the abutment and launched forward on roller bearings.

building a gantry supported at its ends or with beams spanning over falsework. This method of erection is effective for constructing a bridge in sensitive environment or inaccessible areas.

The point of the launching normally begins from one of the bridges where it would be accessible for receiving the prefabricated components. A staging area is required for setting up equipment for bridge launching at one and/or both its approaches. The launch pad must be heavily reinforced from the foundation and the substructure above it. The jacking forces that are to be transferred into the foundation must be designed to account for the erection loads and bearing/Teflon surface friction force. If pulling is included to aid the pushing force, a deadman anchor needs to be designed on the opposite end or at intermediate anchor point. As segments are to be built or assembled behind the launch pad, the bridge with a temporary launching nose attached to it would be launched forward using hydraulic jacks and rigging system. The launching nose serves to reduce the deadweight and supports the front cantilever lead end.

This method of erection is limited to linear or slightly curved movement and cannot compensate for any changes in profile grade during the pushing or pulling operation. Depending on the length of the bridge, it can also be very time consuming with the cycling strokes of the jacks. Challenges include accounting for the reversal of the flexural forces going over and moving off an intermediate support. Friction forces must be accounted for as it is difficult to anticipate the rigidity of the support systems and fabrication tolerances in the structure.

Figure 7.8 illustrates the launching of 13-span continuous curved steel girder across the Risle River Valley, France. Three equal spans of continuous girders were assembled at a time to form 180-m (590-ft.) segments behind each abutment and launched longitudinally in increments of 120 m (394 ft.) forward. The launching nose at the leading edge was guided laterally by jacking at every other pier. Lateral adjustable roller bearings were used to accommodate the lateral movement as the bridge was launched. After the complete superstructure was launched, the deck formwork was rolled forward and cast in defined sequence. The girders were pre-cambered for dead load deflection. The longitudinal launching method of construction is effective for continuous structures across deep valleys, inaccessible terrains, protected waterways, and/or sensitive environment.

7.5.2 Bridge Skidding

To skid or slide, a prefabricated bridge erected adjacent to an existing one transversely is another common method used widely. This method requires the use of heavy-duty roller bearings riding on flat rigid support surface. There are also proprietary skid equipment designed to ride on specially prepared track and it is limited to a linear movement. Small differences in height can be tolerated by changes in the shape of the bridge. Falsework for both the new and old structures is required. The process is time

FIGURE 7.9 New span (left) being moved laterally into position after existing span was skidded off to the side (right).

consuming but can be manageable for a bridge that can be closed for a weekend. This method is effective in a location where long detour is intolerable or the closing of the bridge for a long duration is unacceptable. It is also effective when the existing bridge can be used to carry traffic and thus avoid having to build the temporary bridge near the vicinity. The bridge move is a two-step process—skid the existing bridge out and move the new bridge in. The bridge could be replaced in a matter of hours and usually this is done over a weekend to include finishing the approaches. Either a simple span or continuous bridge can be designed to be skidded into place. With heavy lift equipment, there is almost no limit as to how heavy one can move a bridge. For example, a bridge weighing over 10,000 tons was skidded successfully in the United Kingdom as part of the Channel Tunnel Rail Link.

Figure 7.9 was taken from a successful project under Bundle 401-Rapid Bridge Replacement Project located on the west side of a tunnel near Elkton, Oregon. A new 225-foot concrete girder span weighing 1500 tons on the left picture was built on temporary falsework adjacent to the existing old deck truss on the right. It was being skidded into place using hydraulic jacks mounted on steel tracks. The move took less than 2 hours during a 56-hour weekend traffic road closure but was reopened after 48 hours, 8 hours ahead of schedule. Demolition of the existing span continued offline and without interfering with traffic operation. Containment platform was installed as part of the falsework to prevent falling debris into the salmon spawning beds below. The cost savings of not having to build a temporary bridge is half a million dollars. The conventional construction schedule with staging and maintaining one open lane for traffic 24 hours a day would have taken 6 months. That how attractive ABC is in its ability to deliver at a fraction of the time it normally would to replace a bridge.

7.5.3 Floating Bridges Using Barges

Bridges have been successfully erected on barges and floated into places. The technique of using water to ballast the payload is long recognized by the mariners. High-capacity pumps can quickly add or remove water in barges to float or sink them into place after a structure is positioned over its final location. This method is effective for structures over navigable waterways as barges and tug boats require freeboard for it to access shallow waters. It can be combined with a heavy floating crane to lift a bridge payload off a barge and erect it span by span. It has been an efficient and effective method of construction for long causeway structures in coastal states.

Figure 7.10 shows the Fremont Bridge being floated on the Willamette River in Portland, Oregon. The main steel tied arch span of the Fremont Bridge was erected on steel pilings off Swan Island. It was transferred onto barges and floated upriver. The bridge is a double-decked structure carrying U.S. Route

FIGURE 7.10 Floating the Fremont Bridge on barges on the Willamette River, Portland, Oregon (completed in 1973).

30 and I-405. It has an orthotropic upper deck, which framed directly into the tied girders, and a lower concrete deck, which is supported by floor beams and stringers. The main structure is a three-span, stiffened steel tied arch having a total length of 2152 ft., a record of its time for many years. The center arch span has a total length of 1255.25 ft. and two equal end spans of 448.24 ft.

7.5.4 Lifting/Jacking Bridges

The lifting method involves moving a bridge vertically using either hydraulic jacks or cranes as shown in Figures 7.11 and 7.12. The method is largely place independent; height differences are easily accommodated but overhead wires and crane outriggers must be considered. As heavier lifting and jacking equipment are being developed, this method is by far the most popular method of erection. The offshore construction industry had erected many big bridges over large bodies of water using heavy cranes mounted on barges in Japan, Canada, and Norway. The lifting capacities for constructing the prefabricated elements of the Prince Edward Island Bridge in Canada exceeded the 5000 metric tons range. The posttensioning industry has been developing heavy duty hydraulic jacks that have almost unlimited capacities to raise prefabricated concrete superstructure bridge spans into position. This method of construction remains favorable to the contractors who have access to cranes and hydraulic jacks. The important thing to remember is that the pickup or jacking points should be considered in the design and construction of the elements so as not to overstress or damage them during the lifting. The limit of the tension and compression stresses during the lifting/jacking should also be clearly defined to prevent undesirable cracks in concrete members and yielding or buckling in steel members.

7.5.5 Driving Bridges Using Self-Propelled Modular Transporters

The Europeans have been using SPMT for erecting bridges for many years. The author cochaired a U.S. scan team to Europe and Japan and reported on the use of this technology and promoted its use here in the United States (Ralls, Tang, Bhide et al. 2005). It is becoming a popular method of ABC construction as this technology tends to be suitable for replacing bridges on the Interstate System where the ground is level and improved as shown in Figure 7.13. Several states have successfully erected numerous bridges over weekends and night closures. The driving method allows a bridge to be assembled at a location that is independent of its final position. It is then moved from where it was assembled on the SPMT to its final position of a 100 ft. to a few miles away. This method also has the advantage that height differences are easily adjusted using special jacking and support equipment. The SPMT loaded moving speed ranges from

FIGURE 7.11 Lifting the Fremont Bridge main arch span using strand jacks.

FIGURE 7.12 Full superstructure I-215 east over 3760 south being lifted into place by cranes.

FIGURE 7.13 Moving full superstructure I-80 State to 1300 east using self-propelled modular transporters.

3 to 7 miles per hour. The transporters can be assembled together to form multiple units to spread out the loads, which can exert as much as 1500–2000 pounds per square foot pressure. Ground improvements can be made with conventional grader and compactor or can be steel plated over to form the travel paths.

A single SPMT has either six or four axle lines. Each axle line consists of four wheels arranged in pairs and can support 33 tonnes. Each pair of wheels can pivot 360° rotation about its support point. An SPMT has complete freedom of movement in all horizontal directions. Through its hydraulic suspension system, equal loads are maintained on each axle even on irregular surfaces. The bed of the SPMT can be raised by 600 mm (24 in.) and tilted in both directions to maintain a horizontal bed on an inclined surface. Grades as steep as 8% have been used; maximum grade may vary and is dependent on specific site condition and surface friction between payload and the SPMT. Vertical lifting equipment can be mounted on the SPMT platform if required. The SPMT is self-propelled and can be coupled longitudinally and laterally to form multiple units all controlled by one driver. The driver walks with the units and carries a controller connected to the units by an umbilical cord. The controller has four basic commands: steering, lifting, driving, and braking. The author suggests readers to the FHWA published references (Ralls, Tang, Bhide et al. 2005; FHWA 2007) for more information on the use with SPMT move.

7.6 Structural Elements and Systems

It is been recognized that "elements" are individual prefabricated structural components such that when put together they formed a "system." Like an erector set, a system could be part of the substructure elements, superstructure elements, and/or a complete bridge structure. The intended design and economics of the elements is that they can be repeatable, standardized, and mass produced. Prefabricated elements can be made of fiber-reinforced polymer (FRP) composites, steel, concrete, or combination of two or more materials known as "hybrid" materials system. Depending on the materials, the elements can be fastened mechanically or posttensioned together. They can be built integral with closure pour using high-performance grout or concrete. They can also be bonded together with adhesives and in combination of any structural connection details as appropriate. These elements may be connected transversely or longitudinally; the only restriction may be the limit to their size and weight for handling and shipping. FHWA published a manual containing more than 150 proven details used by various states and readers are referred to Section 7.7 in this chapter for more details.

7.6.1 Hybrid Systems

Bridge engineers have several options to consider when designing hybrid systems. The recently developed high-performance materials such as high- and ultra-high-performance concrete (HPC and UHPC), high-performance steel, FRP composites are the material technology innovations of the twentieth century. They can improve the long-term performance and durability in bridges using prefabricated elements and systems. They are most cost-effective when optimized in hybrid systems or reducing a line of steel or concrete girders altogether. These high-performance materials normally carry an added premium as developers are trying to recoup their development costs. Many ambitious project designers started out in design with the goal of completing a complete bridge structure with only one high-performance material and found them to be too expensive. They have had to scale back their design to meet budget constraints and changed to using hybrid materials to take advantage of each material's strength while balancing cost. There are still untapped potentials for the successful applications of hybrid systems, which applies not only to materials but also to design types like the Hillman Composite Beam system. The author is anticipating this innovation and can be further exploited with the increasing demand for prefabricated elements and systems in ABC projects. With widespread applications, the cost of these innovative materials will be cost-competitive.

7.6.2 Precast Deck Systems

7.6.2.1 High-Performance Concrete Decks

The conventional deck construction requires sequential process from forming deck and overhangs, tying reinforcing steel, placing and curing concrete, stripping formwork, and installing guard rails can take on an extended timeframe. Based on past performance history, the service life of conventional concrete decks lasted about 25 years. Precast deck systems made with HPC and pretopped with a maintainable wearing surface overlay are designed for 75–100-year service life and are much quicker to install.

There are several precast deck systems that are available; some are generic and some are proprietary systems. The early use of partial depth deck panels served as permanent formwork for the concrete pour. This system requires a sequential process to complete the full-depth deck and defeats the goal of speed under ABC schedule. Full-depth deck panels are better suited for ABC method. FHWA is researching the use of UHPC for joining the panels to address reflective cracking.

An existing typical bridge deck built with unprotected mild reinforcing steel can last anywhere from 20 to 40 years before it needs to be replaced. Many deck-girder bridges are due for a mid-life deck replacement due to heavy traffic and use of deicing salts. Sometimes the dead load becomes an issue for an existing or movable bridge; the use of precast and lightweight concrete decks will maintain or improve the live load capacity.

For new bridge construction, performance, durability, and speed become the driving factors for use of precast deck systems. Precast deck systems are popular because they have been proven successful for variable bridge widths in both big and small bridge projects. The panels are cast with blockouts for shear studs that are to be welded to the girder for composite action and then grouted in place. Some applications specify the use of posttensioned strands to connect the longitudinal or transverse segments and keep the deck under compression at all times. Some decks are prefabricated with exposed reinforcement steel to overlap with an adjacent segment; the closure joint pour between segments will make them monolithic.

7.6.2.2 Steel Structural Deck Panels

Open and close steel grid deck panels are proven systems suitable for ABC projects. Steel panels may be suitable for staged construction on a long-span structure when traffic flow is required to be maintained throughout the construction zone. Orthotropic deck systems have been used on several major bridge crossings due to its lightweight and are suitable for ABC application. Steel fabrication process and weld details have been greatly improved to ensure high fatigue resistance for cyclic loadings.

Figure 7.14 shows the use of a concrete-filled gird deck. The bridge connects a remote community in Wallowa County, Oregon, and the State required the contractor to maintain one open lane for traffic at all times during construction. The concrete-filled grid deck supported by steel curved girders was constructed of two halves. The contractor built the first half (right of picture) of the new bridge adjacent to the existing

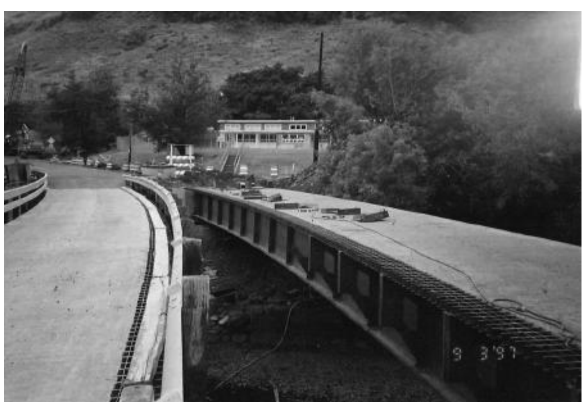

FIGURE 7.14 Use of modular system and concrete-filled grid deck for accelerated bridge construction staged construction. Sequence: Built first half adjacent to existing structure and switched traffic over to new; demolished existing structure and built second half. Skidded two sections together and completed with a closure pour.

structure (left of picture). When traffic was switched over to the new half structure, the existing single-lane bridge was demolished and the other half was built in its place. The final two-lane structure involved some skidding to connect the two halves and finished with a longitudinal concrete closure pour in the middle.

7.6.2.3 Fiber-Reinforced Polymer Composite Decks

The FRP composite deck systems have been used in several bridges with good success. This space age technology is made of high-performance, high-corrosion and high fatigue resistance, and lightweight carbon and/or fiberglass materials, which are highly desirable qualities for bridge application. The deck systems are proprietary products and they have been tested and proven to be excellent application for bridge decks. Some of the early FRP deck failures were caused by improper connection details between the girder and deck panels; some thoughtful consideration and attention should be given to these details. Generally, the use of FRP decks for ABC projects is most effective in movable bridges and long-span trusses. Replacing an existing heavier deck on an existing bridge with a lighter FRP deck would either increase a bridge's live load capacity or restore it to legal loads.

7.6.3 Complete Superstructure Systems

The common ones are the side-by-side deck beams and they can carry traffic immediately after they have been placed. Many owners complained that the thin overlay wearing surface placed over the beams would crack as reflected upward from between the beams. These cracks allow water to penetrate into them. When subject to freeze-thaw cycles, it reduces the service life of the bridge. Improved details like those recommended in FHWA 2009 Reference and new grout mix designs should resolve this problem.

The American Institute of Steel and Iron (AISI) has been working with researchers and practitioners to develop steel girder modular bridges pre-topped with a concrete deck. This system may be suitable for single and multi-span girder system designed as simple span for dead load and continuous span for live load (Tang 2009).

Michigan DOT (MDOT) has built some adjacent double tee-beams prestressed and posttensioned with carbon fiber strands. The concept has proven the use of any typical girder that has a top flange wide enough to form a deck can be designed into a complete superstructure system for ABC projects. End segments should be prefabricated with built-in parapet/guard rails, lighting fixtures, and lane pavement markings to make the superstructure complete when it is fully erected. Other systems such as the inverted tee-beams and voided box beams are already available and when used with the improved connection details will perform satisfactorily. Currently, MDOT and Lawrence Technological University are starting a 3-year pooled fund research project for testing deck bulb tee girders for ABC application. There will be more information available in the near future.

The Minnesota DOT built several bridges using an innovative and fast-tracked precast superstructure slab system similar to the French's Poutre Dalle System (Dimaculangan and Lesch 2010). This precast and prestressed slab is of shallow, inverted tee-beams that are placed adjacent to each other and span between supports. The beams are connected with a longitudinal joint in which 180° bent hooks extended from the sides of the inverted tee-beams, and additional reinforcement provides continuity along the joint. A closure concrete pour is placed between the tee-beam webs and over the top of their stems to form a solid composite cross section. The slab can span up to 100 ft. and it is shallower then precast girders. A closure pour can be done over live traffic below and without the need of forming it. This application is advantageous for many overhead crossings.

There will be more superstructure systems to be developed in the near future—some are fine-tuning the existing elements with improved connections. Others may be of hybrid materials and designs like the Hillman Composite Beam, which is a proprietary design, that utilizes an arch design with concrete, steel, and FRP composite materials, to form a composite-hybrid-beam system. The bridge-in-a-pack is another innovation developed by the University of Maine and the Maine DOT. The precast segmental concrete systems are already in use and more systems and advancements are in progress. Engineers who can exploit the strength of each material and balance out the cost for a complete superstructure bridge system will be well rewarded.

In Oregon, several complete steel superstructure systems in the past have been listed as successful ABC projects. The Multnomah County floated a complete tied arch bridge over the Willamette River near the Sauvie Island. The Oregon DOT floated a complete tied arch main span of the Fremont Bridge and jacked into position in 1973 (see Figures 7.10 and 7.11). ODOT also delivered a project involved with a prefabricated concrete tied arch superstructure adjacent to the existing bridge that was to be replaced and then skid it into place over a 5-day bridge closure. These are some of the examples and there are many more similar projects accomplished in other states.

7.6.4 Substructure Systems

Texas DOT (TXDOT) was the first to exploit prefabricated bent caps under their early ABC projects. From that application, TXDOT has explored bent columns and footings. These successful projects are paving the way for expanding prefabricated elements to include abutment and wing walls. The concrete segmental bridge industry has made great advances with segmental construction and many applications can be quickly brought in line for erecting abutment and wall segments. There should be more effort put in the foundation study to expand the use of shallow foundation, especially for land bridges. As part of the Every Day Count Initiative, the FHWA has been developing and promoting the geosynthetic reinforced soil, integrated bridge system, expanded polystyrene geofoam, and continuous flight auger piles to support ABC. More information on these innovations may be found on the FHWA Web site.

The AISI is coordinating the development of steel pile bent pier systems and cellular structures for ABC. These structures are to be constructed with circular or elliptical steel sheet pile cells and filled with soil. The cells support the soil pressure and the bridge loads through hoop action in the sheet piling. They can be installed efficiently and cost-effectively, offering significant scour protection (Tang 2009).

7.6.5 Complete Bridge Systems

Puerto Rico DOT completed four bridges in San Juan using prefabricated bridge elements from spread footing to deck and guard rails as shown in Figure 7.15. Colorado DOT allowed the contractor to use precast abutment, wing walls, and superstructure to complete the Mitchell Gulch in less than 48 hours. New Hampshire DOT was successful in building a complete bridge system with spread footing, abutment and wing walls, and deck beams near Epping. More states' DOT are under pressure to maintain traffic flow during construction and will be considering complete bridge system that can be delivered over a long weekend using prefabricated elements. It has been proven with projects completed in several states' DOTs, especially in Utah, that designing and erecting a complete prefabricated bridge is not only possible but cost-effective when cost of users delay is considered.

7.7 Connections Details

The author was involved with a research project that led to the publication of the FHWA-IF-09-010, Connection Details for Prefabricated Elements and Systems (FHWA 2009). This document contains more than 150 connection details that are recommended for PBES applications. They represent the state-of-the-practice used by several states' highway agencies. The author suggests readers to start with the guidance published in this public document for details specific to their applications.

There is a concern for the lack of available connections between bridge elements in high seismic zones. The American Concrete Institute provided guidance for emulative detailing per ACI 550.1R-2 and ACI 318. The use of grouted reinforcing splice connectors is an acceptable alternate method of connecting the reinforcing steels for continuity. Its structural performance is equivalent to and treated as a CIP monolithic reinforced concrete. The grouted splice connectors can transfer axial, shear, and

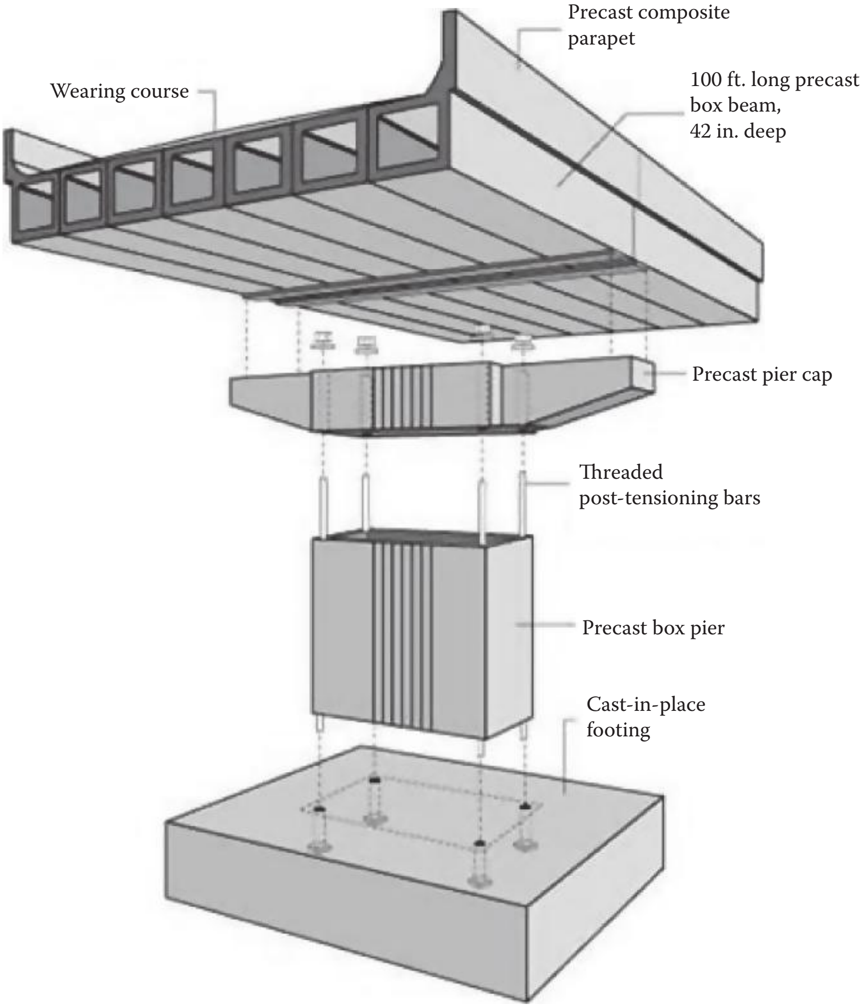

FIGURE 7.15 Complete prefabricated bridge elements to form a bridge system used in Puerto Rico's Baldorioty Project.

moment forces effectively. Several bridge projects have successfully used the grouted connectors to connect bent/pier cap to column, column base to footing, and wall to footing.

Washington DOT (WSDOT) and the University of Washington did a study and developed a force-based and displacement-based seismic design procedure for CIP emulation and hybrid precast concrete piers. The expected damage to piers designed with the procedures in a design-level earthquake was first estimated. Then after testing the results, the study showed the force-based and displacement-based design procedures were found to produce designs that are not expected to experience an excessive amount of damage in a design-level earthquake. The evaluation considered three types of damage to the pier columns, that is, spalling of the cover concrete, buckling of the longitudinal reinforcing bar, and fracturing of the longitudinal reinforcing bars. A designer who follows the design procedures as recommended in the study will not be required to perform nonlinear analysis for WSDOT bridges (Wacker, Hieber, Stanton, and Eberhard, 2005).

The U.S. scanning team reported that the Japanese engineers have designed and built several bridges using prefabricated elements. The key to their success in detailing for seismic load is with field closure pours to make them monolithic. For example, the deck panels are detailed with hairpin or looped bars placed overlapping in the invert "tee" joint; the closure pour does not require forming because the flanges served as the formwork. Another example is the partial prefabricated substructure panel system that allowed the preassembling and monolithic casting of the joints, which completed the partial system in its final position without requiring further formwork. This concept may be applied to almost any system requiring monolithic behavior in high seismic zones.

7.8 Environmental Issues and Mitigations

All bridge designers should be familiar with environmental laws and mitigations when it comes to bridge work. Whether it has to do with short construction season in Alaska or reduced work window for in-water work due to fish spawning in the Pacific Northwest, ABC may be the best option for complying with the schedule related to the environment or its constraints. Environmental laws are often more restrictive to minimizing the disturbed footprints of construction sites, and ABC bridges help reduce the potential impact to an area by launching bridges from one end to the other. The FHWA Decision-Making Framework mentioned in the preceding discussion listed several environment-related questions for the designer to consider what mitigations to take for moving an ABC project forward. When working in a sensitive environment and if bridge construction activities are in the critical path, several proven ABC techniques are available to deliver a project and mitigations can be alleviated. ABC projects may reduce permitting process and save on both time and expenses associated with environmental studies and mitigation reports. Reduced right-of-way acquisition to stay out of sensitive environments becomes less contentious. ABC enables the owners to deliver projects quickly, save time and cost, and enjoy the accolades of the communities on either side of the bridge.

When one speaks of environmental issues, generally one refers to the natural environment, wildlife habitat, endangered species, and aquatic plans. The transportation and highway infrastructure renewal work in the twenty-first century will expand its scope to involve a good part in the urbanized areas. Land acquisitions and development, community livelihood, noise and dust pollutions, business impacts, and mobility are other forms of environment-related issues that will require some form of mitigations when working in urban areas. Night work during off-peak hours may be restricted due to noise and ABC becomes the method of choice. Stakeholders will find ABC a great advantage to alleviating impacts and the demand for accelerated project delivery will be increasing as owners are looking for more efficient and effective means to maintain and operate the existing infrastructure.

7.9 Inspection and Maintenance

In the American construction culture, it is understood that the contractor delivers as per the contractual requirements as established by the owner who ultimately has the sole responsibility to maintain the safe operation during the life of the structure. The owner considers the accessibility, inspectability, and maintainability of the bridge regardless of how it is to be built. Details must be provided for the contractors to be able to build the bridge. If components are not accessible for inspection or for repair, then it is an unacceptable design. Owners are required to load-rate their bridges and design calculations and detailed drawings are maintained through the service life of the bridges. The management of these records for future reference should be no different than conventionally built bridges.

Several states' DOTs have standard construction specifications for handling, inspecting, and maintaining prefabricated bridge elements. Concrete precast plants are required to be certified under a third party like Precast/Prestressed Concrete Institute that administers a plant certification program. This will ensure that the industry best practices are being followed.

Designers may need to establish a unique bid item and specify quality assurance and quality control plan if it is not adequately covered under the standard specifications. The owner may require the contractor to submit the names and qualifications of key personnel and the plant's process for tracking material certifications. The owner may also require a contractor to identify pick-up points and shipping method.

Inspection of precast concrete elements will be required during both the precasting operation and field placement. Acceptable tolerances should be specified in accordance with the PCI standards. Acceptable repair procedures should be well defined to avoid claims. When an element is being rejected for any reason, the ability to deliver per project schedule will be greatly impacted since there is not enough time to refabricate it. The stakes are high for ABC project and quality and damage control must be put in place to avoid these problems.

Similarly, steel bridge fabricators are required by most owners to be certified under the American Institute of Steel Construction under its appropriate category. The AASHTO steel construction specifications cover steel fabrication and they are generally incorporated into the contract documents.

7.10 Future Development in ABC

ABC has been actively promoted and implemented by the FHWA, TRB, and AASHTO in the past 10 years. States are implementing ABC projects because it is the right thing to do in terms of providing customer services and mobility in support of the growing demand placed on the surface transportation networks. At the time of this writing, there are several research projects underway and the author provided a list of them for the reader to track. Some discussions and their research scope and goals are included when information is available. Since the FHWA is taking the lead in promoting and implementing ABC, they would be the best source for information being made available to the public.

Obviously, for ABC to continue, its success, standard designs, and specifications should be developed and shared openly. The author acknowledges the ongoing work funded under TRB, Strategic Highway Research Project II (SHRP 2) RO4 to begin developing standards and specifications is headed in the right direction. The FHWA, Highway for Life projects, using UHPC for the closure pours in joining the deck panels and other precast components will improve the connection details and long-term performance of bridges using PBES.

Training is probably one of the most important drivers to advancing ABC. Without training, designers are reluctant to venture out at their own risk. The National Highway Institute under FHWA has developed and implemented training for ABC under the FHWA Every Day Count initiative. The Florida International University ABC Center has been conducting webinars on ABC technology. The FHWA is in the process of developing a design manual on ABC, which will serve as a best practice guide for designers using ABC. The author hopes that training should be part of this development as it will allow the state DOTs to develop more ABC projects in serving the public.

7.11 Summary

Maintaining mobility is a high priority of the nation's transportation leaders for the American economy. In the rebuilding of an aging highway infrastructure in the United States, it has created an opportunity to bring out the best of every bridge engineering innovations moving into the twenty-first century. ABC concept is one innovation that has been proven over and over again to enable bridge owners to deliver projects in a fraction of the time what it normally takes otherwise using traditional method. Several ABC strategies are already in place and those who are using them not only found them useful but also improved on them for future projects. The use of PBES is one that has been a popular strategy with great success because there is already a well-demonstrated history of their performance. The benefits of ABC from enhancing the environment, mobility, and safety are well recognized. Those who implemented ABC projects are finding that they can deliver efficiency and cost-effectiveness with their designs. These are the main reasons for the FHWA, AASHTO, and TRB to continue developing and promoting PBES. There are several methods of construction that are listed here and most of them are several decades old but remain valid today. Heavy-duty lifting equipment and special transporters are accessible to any contractor who wants to use them to accelerate project delivery. Connection details and design guides are available and many of them are public documents that can be obtained through the FHWA free of charge. Continuing research is targeting more standard developments and applications. Whatever one needs to advance and implement ABC technology, information is readily available and this chapter provided several recommended sources. This accelerated bridge technology is one great innovation and is creating exciting challenges for the civil engineering profession moving into the twenty-first century.

Defining Terms

AASHTO: American Association of State Highway and Transportation Officials
ABC: Accelerated bridge construction
ACI: American Concrete Institute
AHP: Analytic Hierarchy Process (ABC Decision Tool)
DOT: Department of Transportation
FHWA: Federal Highway Administration
ISTEA: Intermodal Surface Transportation Equity Act
NCHRP: National Cooperation of Highway Research Program
PBES: Prefabricated bridge elements and systems
SAFETEA-LU: Safe Accountable Flexible Efficient Transportation Equity Act: A Legacy for Users
SPMT: Self-propelled modular transporter
TEA-21: Transportation Equity Act for the 21st Century
TRB: Transportation Research Board
TTI: Texas Transportation Institute

References

AASHTO. 2010a. *AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, 5th Edition, with 2010 Interim Revisions*, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2010b. *AASHTO LRFD Bridge Construction Specifications, Customary U.S. Units, 3rd Edition, with 2010 Interim Revisions*, American Association of State Highway and Transportation Officials, Washington, DC.

Battles, S. 2004. "Congestion is the Challenge," *Public Roads* May/June 2004, Volume 67, No. 6. http://www.fhwa.dot.gov/publications/publicroads/04may/07.cfm (accessed October 31, 2011).

Dimaculangan, M. C. and Lesch, T. 2010. "Minnesota's Precast Composite Slab Span System," *ASPIRE Concrete Bridge Magazine*, 2010 Issue, pp. 40–42.

Doolen, T. 2011. *Accelerated Bridge Construction (ABC) Decision Making and Economical Tool, TPF-5(221)*, FHWA Transportation Pooled Fund Program, http://www.pooledfund.org/Details/Study/449 (accessed October 31, 2011).

FHWA. 2005. *ACTT Manual: A "How To" Guide for State Highway Agencies* Federal Highway Administration, Washington, DC, http://www.fhwa.dot.gov/construction/accelerated/if05038.pdf (accessed October 31, 2011).

FHWA. 2006. *Decision-Making Framework for Prefabricated Bridge Elements and Systems (PBES), FHWA-HIF-06-030*, May, Federal Highway Administration, Washington, DC.

FHWA. 2007. *Manual on Use of Self-Propelled Modular Transporters to Remove and Replace Bridges, FHWA Publication FHWA-HIF-07-022*, June, Federal Highway Administration, Washington, DC.

FHWA. 2009. *Connection Details for Prefabricated Bridge Elements and Systems, FHWA Publication FHWA-IF-09-010*, March 30, Federal Highway Administration, Washington, DC.

FHWA. 2011a. *Work Zone Safety Fact Sheets*, Federal Highway Administration, Washington, DC, http://safety.fhwa.dot.gov/wz/facts_stats/ (accessed October 31, 2011).

FHWA. 2011b. *Accelerated Bridge Construction*, Federal Highway Administration, Washington, DC, http://www.fhwa.dot.gov/bridge/abc/ (accessed October 31, 2011).

MADOT. 2011. *Accelerated Bridge Construction*, Massachusetts Department of Transportation, Boston, MA, http://www.eot.state.ma.us/acceleratedbridges/ (accessed October 31, 2011).
Ralls, M. L., Tang, B. Bhide, S. et al. 2005. *Prefabricated Bridge Elements and Systems in Japan and Europe, FHWA Publication FHWA-PL-05-003*, January, Federal Highway Administration, Washington, DC.
Schrank, D., Lomax, T. and Eisele, B. 2011. *2011 Urban Mobility Report*, University Transportation Center for Mobility, Texas Transportation Institute, College Station, TX, http://mobility.tamu.edu/ums/report/(accessed August 23, 2013).
Tang, B. 2009. Notes captured from presentations when he participated in a follow-up meeting on Modular Steel Bridge Workshop that was sponsored by the AISI and FHWA, Baltimore, MD, August 12.
USDOT. 2012. *DOT Strategic Plan, Fiscal Years 2006–2011, Reduced Congestion Strategic Goal*, United States Department of Transportation, Washington, DC, http://www.dot.gov/dot-strategic-plan (accessed October 31, 2011).
Wacker, J. M., Hieber, D. G., Stanton, J. F. and Eberhard, M. O. 2005. *Design of Precast Concrete Piers for Rapid Bridge Construction in Seismic Regions, Report No. WA-RD 629.1*, August, University of Washington, Seattle, WA.

Further Reading

Blanchard, B. A., Bohulav, T. R., Schneider, C. et al. 2009. "Best Practice in Accelerated Construction Techniques," *Scan Team Report, NCHRP 20-68A, Scan 07-02*, Transportation Research Board, Washington, DC.
Oregon DOT. 2011. *Bridge Design and Drafting Manual, Section 1.1.2.9.12*, Oregon Department of Transportation, Salem, OR, http://www.oregon.gov/ODOT/HWY/BRIDGE/docs/BDDM/apr-2011_finals/section_1-2004_ap11.pdf (accessed August 23, 2013).
Shahawy, M. A. 2003. *Prefabricated Bridge Elements and Systems to Limit Traffic Disruption during Construction, NCHRP Synthesis 324*, Transportation Research Board, Washington, DC.

Relevant Websites

http://www.fhwa.dot.gov/hfl/
http://onlinepubs.trb.org/onlinepubs/nchrp/docs/NCHRP20-68A_07-02.pdf
http://www.fhwa.dot.gov/bridge/abc/
http://www.fhwa.dot.gov/bridge/prefab/
http://www.fhwa.dot.gov/construction
http://www.ops.fhwa.dot.gov/, Full report on Traffic Congestion and Reliability
http://www.udot.utah.gov/
http://www.wsdot.wa.gov/eesc/bridge/ABC/

8

Bridge Construction Supervision and Inspection

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8.1 Introduction

Bridge construction supervision and inspection ensures that bridge projects are built by a contractor in compliance with the contract documents, including code requirements of federal, state, and local governments, such as *AASHTO LRFD Bridge Construction Specifications* (AASHTO 2011), Caltrans *Standard Specifications* (Caltrans 2010), MOTPRC *Highway Bridge and Tunnel Construction*

Specifications (MOTPRC 2011), special project provisions, and construction plans, throughout the construction process. Caltrans (2012a) *Construction Manual* provides detailed guidelines for bridge construction. AASHTO/AWS D1.5 *Bridge Welding Code* (AASHTO/AWS 2010) is applicable for steel bridge fabrication in the United States.

The service scope of bridge construction supervision and inspection including project scope and working scope should be clearly defined in the bidding documents and the supervision contract.

Project scope usually covers the permanent structures and temporary structures. For the construction of a large steel suspension bridge, permanent structures include substructures such as foundations consisting of piles cap, piles, piers and towers, anchorages and abutments and superstructures consisting of steel box girders/trusses girders, main cables, suspenders, steel anchor box, suspenders, maintenance cart, dehumidification system, subsidiary steel components, and various embedded parts. Temporary structures supporting the permanent structures during their erection and construction may include trestle, tower, bracing, inhaul cable fixed system, scaffold, and forsework.

In modern bridge construction, large-scale, standardized, and prefabricated bridge elements and systems, such as precast concrete segments, steel orthotropic girders, and steel–concrete composite girders, are widely used. For larger prefabricated elements, steel elements in particular, accuracy of structure shape and size, welding procedure and quality, splices and connections, corrosion protection, and so on, are always the keys and difficult points of steel structure fabrication and erection. Therefore, the inspection of steel projects is often performed by experienced professional supervision institutions.

Working scope involves field inspection, quality assurance (QA), change order management, and contract administration.

Bridge construction supervision is a complex discipline that requires integrating construction techniques and management skills with the art of communication. A bridge supervision engineer (SE) or resident engineer (RE) must not only have knowledge and experience of bridge design, construction, and risk management, but also have knowledge and experience related to social, economic, and political fields.

This chapter presents supervision and inspection principles and guidelines and guidelines for inspecting materials, construction operations, component construction, and temporary structures. It also touches safety considerations and documentation.

8.2 Objectives and Responsibilities

The objective of bridge construction supervision and inspection is to ensure that a project is constructed in accordance with project plans, construction specifications (AASHTO 2011, Caltrans 2010, MOTPRC 2011), and special project provisions on time and within budget. Project plans are a set of drawings or two-dimensional or three-dimensional diagrams used to describe a bridge object and to communicate building or fabrication instructions. Construction specifications and special project provisions describe the expected quality of materials, standard methods of work, methods and frequency of testing, the variation or tolerance allowed, and expected project delivery schedule.

The primary responsibility of an SE is to ensure that bridge structures are constructed in compliance with project plans and specifications and to ensure that the construction operations and/or products meet the safety, environmental, cost-effective, and quality standards. The SE is also frequently known as an RE, or quality assurance inspector (QAI), who is the owner's representative. In this chapter, SE will be used for simplicity. The SE is also responsible for determining the design adequacy of temporary structures proposed for use by a contractor. The qualified SE should have a thorough knowledge of construction specifications and clients' requirements and should exercise good engineering judgment. The SE should keep a detailed diary of daily observations, noting particularly all warnings and instructions given to the contractor. The SE should maintain continuous communication with the contractor, provide recommendations, and resolve issues before they become problems. The general role of an RE in the United States for projects for both public and private sectors, including the definition; qualifications; duties; responsibilities; authority and liability; and professional aspects consisting of contract administration, project organization, cost control, QA, and ethics communications, are discussed in American Society of Civil Engineers proceedings (Re and McKittrick 1985).

A supervision contract is the agreement of supervision work between the supervision service company and the project owner. The legal responsibilities of the supervision are the basis of a supervision contract and shall not be violated.

8.3 Core Supervision Tasks

Bridge project risks exist in each process and each step of bridge construction. Risks in fabrication, manufacture, erection, and assembly stages mainly come from the early stages of project development such as project feasibility analysis, development of specifications and standards, survey, design, and design review. And the quality problems left in construction stages will in turn become the main risk source of the operation stage. In fact, most bridges are designed properly, but they may not always meet the final quality requirements. The key to a successful project is quality control (QC) of the whole construction process including construction planning, scheme, techniques, management, staffing, equipment, materials, construction environment, and so on.

These problems may likely come up in bridge construction: "act first and report later" and not following the procedure, using unreviewed techniques and technology, unqualified quality inspection, unapproved and uninspected material substitution, cheating on workmanship and materials, unprofessional work, unapproved design changes, lack of subcontractor control, and lowering standard after a problem is found.

The business attribute of bridge project supervision is risk management and control, and the core task is safety and QC of the bridge construction project.

8.4 Supervision Service Organization and Its Internal Management

8.4.1 Supervision Service Organization

For a large bridge project, a supervision service organization chart is usually formed, as shown in Figure 8.1. Under the chief supervision engineer (CSE), there are four departments: (1) administration management, (2) engineering technology, (3) testing, and (4) health, safety, and environment (HSE) management. Black dots indicate individual supervision and inspection teams such as civil works, concrete, prestressing, steel structure fabrication, nondestructive testing (NDT), inspection team, and so on.

The CSE is fully responsible for overall project construction supervision. The CSE will provide leadership and direction to the project supervision team for all technical areas during bridge construction. The main responsibilities of a CSE are as follows:

- Supervise at site the contractors' performance and confirm that the bridge is built in compliance with the technical specifications, safety codes, and required quality.
- Develop supervision plans and procedures.
- Oversee and direct supervision staffs.
- Interpret and explain plans and contract terms to engineers in the project and the client.
- Support the contractor to obtain all necessary permits from the client and other stakeholders.
- Take actions to deal with the results of delays, bad weather, or emergencies at construction sites.
- Liaise with supervision staffs, contractors, design engineers, and the client to discuss and resolve matters such as work procedures, complaints, and construction problems.
- Investigate damage, accidents, or delays at construction sites to ensure that proper procedures are being carried out.
- Evaluate construction methods and determine cost-effectiveness of plans.

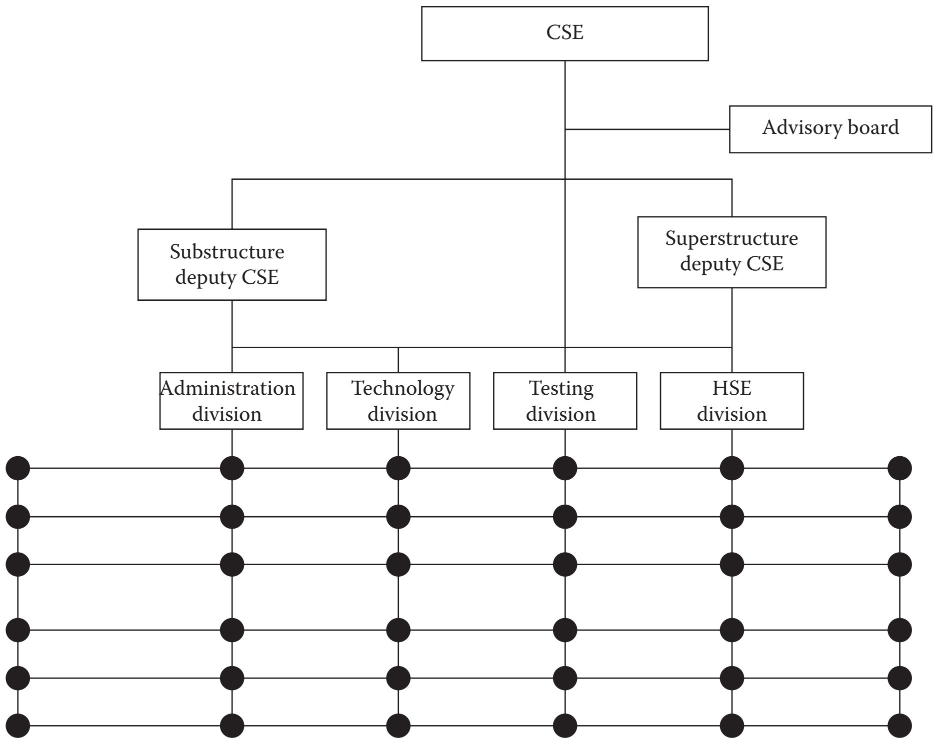

FIGURE 8.1 Supervision service organization chart.

8.4.2 Internal Management

Effective management of a supervision service should establish standards, policies, regulations, and best practices including but not limited to the following:

- Periodic supervision and inspection report
- Contact persons
- Regular meeting
- Special meeting
- Process inspection
- Shop drawing review
- Concealed project inspection
- Emergency response
- Safety protection
- Testing report and review

8.5 Quality Assurance

8.5.1 Main Tasks

QA refers to the planned and systematic activities implemented by the SE to verify that the final product satisfies contract requirements. QC encompasses the activities undertaken by the contractor to ensure that a product is provided that meets contract requirements. QA functions not only emphasize the review of QC work but also include independent inspections. QA by SE is not a substitute for a contractor's QC (AASHTO/NSBA 2002).

QA usually covers three stages: bidding, preparation, and execution. At the bidding stage, the tasks of QA are as follows:

- Review and evaluate the qualification of the contractor so that the contractor team has sufficient ability to ensure the quality of a construction project.
- Answer bidder's enquiries to better serve the needs of contractors who bid for the project.
- Develop amendments to special project provisions.

At the preparation stage, main tasks are as follows:

- Review construction team and personnel.
- Review construction plan including QC, safety, equipment and emergency response, and so on.
- Review and approve shop drawing.
- Inspect material sources and certifications.

At the execution stage, main tasks are to perform standby inspections and sample testing for each operation and process.

8.5.2 Main Activities

QA activities involve verification tests, measurements, inspection, or observations to ensure that construction items conform to the contract requirements.

8.5.2.1 Verification Measurements

Verification measurements have to be conducted for the following items:

- Fabrication details
- Original ground elevations
- Layouts, alignments, and dimensions
- Final project acceptance measurements

8.5.2.2 Verification Tests

- Perform material source inspection to determine whether they meet the material requirements.
- Review certifications of conformance and test reports for materials, products, and members ordered by the contractor prior to purchases. When necessary, perform audit visits to fabrication and manufacture shops.
- Perform sample testing for purchased materials and products per specifications, and reject any unqualified materials and components.
- Perform random sample testing for purchased materials and components during construction.
- Inspect conditions for material stack and storages at any time.

8.5.2.3 Standard Tests

The contractor is required to perform standard material tests and submit the results to the SE for review and approval. SE should observe the overall process of the tests and perform effective standby inspection. SE also performs independent standard tests to verify the parameters obtained by the contractor.

8.5.2.4 Sampling Tests

Sampling tests should include materials, NDT/welding, and painting. SE should check sampling frequency, method, and process performed by the contractor at any time:

- In addition to contractor's QC testing at the frequency specified in the specifications, SE should independently conduct sampling tests at frequencies of 10%–20% to identify the reliability of the contractor's test results.

- When a standby or field SE has questions about material quality and a requirement, the supervision central laboratory should carry out sampling tests immediately. When necessary, the contractor should also be asked to increase the sampling frequency.

8.5.2.5 Acceptance Tests

Acceptance tests are to evaluate actual quality of finished works. SE performs the following tasks:

- Review and supervise the frequency, sampling method, and testing process of core drill sampling testing conducted by the contractor.
- Review and approve the testing plan, equipment, and method used by the contractor upon requirements of the technical specifications. Inspect and supervise test implementation in site, and evaluate the test results.

8.5.2.6 Process/Procedure Assessment

SE should evaluate critical fabrication processes including welding and coating procedures before fabrication:

- Require the contractor to submit a fabrication plan and detailed procedures for review and approval.
- The contractor is required to submit at least two alternatives for the following operations: equipment usages of process testing, operation staffing, materials, construction procedures, embedded observation, and operation method.
- SE should perform standby supervision during the whole process for contractor's process testing and document details.
- The contractor should submit the testing report to SE for review and approval.

8.5.2.7 On-site Standby Supervision

On-site standby supervision and inspection is one of the most important tasks of QA. It is done to continuously monitor major construction processes including evacuation, backfilling, pile driving, concrete pouring, prestressing, and erection. Major tasks include the following:

- CSE shall visit the construction site periodically to find and solve quality problems.
- SE performs standby inspection for concealed work, important components, major fabrication processes, and procedures to eliminate potential effects on quality before they occur.
- SE shall perform inspection and certification immediately after the completion of each process and monitor the contractor's sampling and document operation details.
- SE should observe, find potential effects or risks on project schedules and quality, and report to the CSE and the owner.

8.5.2.8 Subproject Acceptance

- After one subproject is completed, the contractor's quality control inspector (QCI) must perform a systematic check one more time, document inspection records of every operation and results of measurements and sampling testing, and submit the final completion report for acceptance. SE should reject the final report if it does not include adequate and complete information.
- SE should also conduct a systematic verification inspection to accept the completed subproject. When necessary, a measurement or sampling test should be performed. SE should submit the qualified subproject to CSE for signing the Subproject Acceptance Certificate. No further construction work shall be executed if the subproject is not granted the Subproject Acceptance Certificate or the final completion report is not submitted.

8.5.2.9 Field Actions for Nonconforming Materials and Workmanship

During the construction process or after project completion, if SE or QAI discovers some nonconforming materials and workmanship the following actions are performed:

- For nonconforming quality issues at early stages, the filed SE shall bring any nonconformance to the attention of the contractor immediately on discovery. More serious deficiencies are reported to CSE or the owner. However, do not direct corrective actions. If the contractor fails to take corrective action or operates in an unacceptable manner, document the event, and notify QCI that the work may be rejected and contact CSE or owner.
- For obvious deficiencies, notify the QCI verbally first. If a halting operation must be recommended, notify the contractor in writing. Before taking this action, SE must discuss the situation with QCI and obtain direction from CSE. The contractor cannot resume work until the corrective action is made.
- For a deficiency discovered during an operation or after completion of a subproject, which will affect the quality of subsequent operations and the project, SE should request the contractor to submit a written proposal documenting the situation and proposing actions to address this issue for review and approval. A repair or a subsequent operation cannot be performed without SE's approval.
- For a deficiency discovered during the quality guarantee period after project completion, SE should instruct the contractor to repair, strengthen, or rework.

8.5.2.10 Repairing and Strengthening Deficiencies

- For construction-induced deficiencies, the contractor submits a repairing and strengthening proposal for SE's approval. For design-induced deficiencies, the repairing and strengthening method shall be developed by the owner and conducted by the contractor.
- Repairing and strengthening should not reduce quality levels and acceptance criteria and should be an industry-accepted good engineering practice.
- If a deficiency does not affect the safety of the completed project and also meets the design and service requirements, it may not be repaired with the owner's approval. For a construction-induced deficiency without repairing, the owner may negotiate with the contractor to reduce payment for the project.

8.5.2.11 Quality Incident Response

A major nonconforming quality issue not meeting the technical specification requirements during construction including quality guarantee period should be considered as a quality incident. The following responses shall be made:

- Instruct the contractor to pause construction of the project and take effective corrective action immediately.
- Notify the contractor to submit the quality incident report and report to the owner as soon as possible. The quality incident report should include project name, location, incident reason, response plan, and costs in detail.
- Form a special SE committee to visit the incident site to diagnose, test, and evaluate problems. The committee shall review, revise, and approve the incident response plan submitted by the contractor.
- SE should provide resolutions for controversial quality incidents based on careful reviews of all related construction records, design data and hydrogeology information, and additional inspection and testing if necessary. The resolution shall clearly identify the causes and liability of the incident, response cost, and payment method.

8.5.3 Methods and Procedures

Audit fabricators' technical staffing, production capacity, equipment status, and testing facility. Most importantly, examine and review the contractor's QC plan and system, check the effectiveness of QC during operation, and provide suggestions for improvement.

1. Review fabricator's documentations of manufacturing and processing to divide the whole process into smaller processes to determine QA points. Three QA points, R point (review point), W point (witness point), and H point (holding point), are usually adopted:
   a. R point (review point): the QA point that needs to conduct document review is known as review point. SE reviews documents (such as material certificates, inspection reports, etc.) submitted by the contractor or the fabricator at any moment.
   b. W point (witness point): the QA point that needs to conduct standby supervision for complex key operations, testing, and experiments is known as witness point. For W points, the contractor or the fabricator must notify SE for standby inspection. SE should arrange W point inspection to the best of his or her ability. However, the contractor does not need to wait until SE comes to the site.
   c. H point (holding point): the QA point that is an important operation, a concealed project, or a key acceptance test that must be conducted under SE's supervision and certification is known as holding point. For H point, the contractor must notify SE for supervision. SE must arrange site supervision. No H point construction activity shall be conducted without SE's standby supervision.
2. Typical QA procedures are document review, process observation, standby supervision, sampling check, test witness, and so on.
3. Establish the regular meeting system and periodically organize coordination and special meetings to solve problems and coordinate various relationships.
4. Establish the signing and affirming system; clearly identify roles and responsibilities of stakeholders in accordance with the contract documents. Most importantly, SE's roles and responsibilities shall be determined by the signing and affirming system.
5. Establish the QA point reporting system, and ensure that various QA points are performed under SE's supervision.
6. Establish the nonconforming work procedure, and strictly follow it.

8.6 Project Delivery Assurance

One of SE's tasks is to ensure project delivery in accordance with the construction documents. SE reviews the contractor's general project and subproject delivery plans and schedules, monitors work progress, and takes effective approaches to ensure that the contractor's actual progress meets the expected delivery requirements.

8.6.1 Main Tasks

The main tasks in project deliver assurance are to evaluate, analyze, and compare actual project delivery and progress tables and charts with planned delivery schedules; find problems; and take necessary actions to achieve the progress target:

1. The following issues shall be addressed during review and approval of the contractor's monthly/subproject delivery plan and review of the contractor's quarterly, annual, and general plans:
   a. Are critical paths correct to achieve the target delivery schedule?
   b. Are arranged man power and construction equipment enough to complete the work?
   c. Does current construction equipment match each other and keep good?

   d. Is work space enough to perform the work?
   e. Is the transportation plan adequate?
   f. Can spare components meet equipment maintenance requirement?
   g. Is there a construction equipment plan? How long is a maintenance period?
   h. Is there enough time arranged for completion acceptance and field cleaning?
   i. Are materials and man power supply plan adequate to meet delivery requirements?
   j. Subcontract work plan (if applicable).
   k. Temporary work plan.
   l. Delivery and acceptance plan.
   m. Construction environmental and technical problems, which can affect progress.
   n. If necessary the contractor can be asked to reconsider related problems, or discuss with the contractor to reach a consensus agreement.
2. The contractor shall develop and submit the stage delivery and progress plan, subproject construction plan, workforce plan, machinery and equipment plan, and material purchasing plan.
3. Establish an effective communication pattern with the contractor and the owner to ensure expected project delivery targets.
4. Closely monitor project progress; compare actual progress with planned progress, analyze delay reasons, and notify the contractor to take effective action to achieve the delivery goal. The contractor must submit a revised delivery plan for SE's review. When the contractor cannot complete the project or the project is delayed significantly due to the contractor's violation of the contract confirmed by SE, the owner can terminate the contract.

8.6.2 Methods and Procedures

Project delivery assurance must be effectively implemented throughout the whole construction process. The following four steps are keys to ensure the project delivery within schedule: prereviewing, monitoring, postcontrolling, and network planning.

8.6.2.1 Prereviewing

The contractor should submit the general construction plan including the general project delivery plan to SE during the specified period of time after signing the contract. The general project delivery plan usually consists of key paths and main work bar charts. SE should carefully review and analyze rationality and reliability of the project delivery plan. The following issues should be addressed during the review and approval process:

- Does the project delivery plan meet the requirements of the contract document? Are there any potential conflicts and gaps between subproject delivery plans?
- Are subproject delivery plans consistent with the general project delivery plan? Is the project delivery plan compatible with the external environment such as land procurement and structure demolishment?
- Are the contractor's machineries and equipment the same as those in the original bidding document? Are equipment and machineries suitable to topography, landforms, geology, hydrology, and project conditions in the construction site? Are machineries and equipment in good shape? Is the repair and maintenance plan implementable?
- Is the contractor's staffing personnel, including technical personnel, administration personnel, testing personnel, surveyors, equipment operators, and maintenance personnel, the same as that in the original bidding document? Can the staffing arrangement guarantee the project delivery as planned?
- Is the construction of roads, water, electricity, and other temporary facilities properly addressed?

- Are material suppliers confirmed? Can the quantity of inventory material meet the project demands?
- Do the contractor's field laboratories including test equipment and staffing meet the project demands?

8.6.2.2 Monitoring

During the construction process, SE shall arrange regular project meetings to evaluate contractor' project status reports on the implementation of the project delivery plan. For a potentially delayed project, SE shall notify the contractor to take proper and effective actions such as working overtime, double-checking all dependencies and time-constraining activities, crashing schedule, fast tracking, increasing equipment, and improving process to get a slipping project back on track.

8.6.2.3 Postcontrolling

For a delayed project, SE should require the contractor to investigate the reasons of delay carefully and develop an accelerated workable proposal and submit it for SE's review and approval. All delay-added expenses shall be borne by the contractor. The contractor may also be required to pay a penalty for the delayed project as per the contract terms and conditions. The new project delivery schedule shall be negotiated among SE, the owner, and the contactor.

8.6.2.4 Network Planning Technique

The network planning technique control method may be used to analyze project delivery control. In this method, actual construction and delivery time for each project is recorded, compared, and checked in a network diagram. The following are the main cases:

- In case the construction time for a project on the critical path is longer than planned, SE must require the contractor to take effective actions to speed up the critical-path project to avoid any potential delay of the whole project.
- In case the construction time for a project on the critical path is shorter than planned, SE assesses the actual progress and determines whether the project needs to be finished ahead of schedule, and notifies the contractor any owner's decision. For all circumstances, the contractor should be required to revise the network plan and recheck changes related to the critical path to ensure the realization of project delivery goal.
- In case the construction time of a project on the noncritical path is longer than planned, if there is time for justification it generally does not affect the entire network planning. However, if there is no time for justification its effects on the critical path must be evaluated. If this noncritical path changes to the critical path, SE should require the contractor to take corresponding measures to shorten construction time for this project to guarantee the completion of the critical-path project within planned requirements.
- In case the construction time for a project on the noncritical path is shorter than planed, SE should remind the contractor to relocate the working force and resources of the project to the critical-path project to shorten the construction period of the whole project.

8.7 Cost Control Audit

Bridge construction projects are among the largest and most complex financial expenditures undertaken by many entities. It is critical and highly advisable for construction cost control and audit to take place to ensure that the construction costs billed by the contractor and the payments made by the owner are appropriate without affecting project delivery, quality, and operation safety. Cost control and audit is a continuous activity of monitoring actual expenditures and payment for each project and work through organizational measures, technical measures, economic measures, and contract measures. It is very important for SE to collect timely, accurate, reliable, and complete cost-related information for the project.

8.7.1 Principles

"General principles" are overall control, critical control, weekly report, monthly summary, project completion summary, and billing. Records include item accounting table, intermediate quantity list, monthly quantity summary, item completion summary, and final completion bill.

The "contract principle" is that cost control and audit must be conducted by methods and procedures specified in the contract documents.

The "technical principle" is that a reasonable construction and processing scheme should be selected to reduce technical cost.

8.7.2 Main Tasks

1. Sign and issue mobilization advance payment certificate.
2. Verify quantities and values of completed work items on the project site per the checklist in the contract document.
3. Review, sign, and issue interim payment certificates and any payment certificates of terminated contracts per the contract document. Refuse payment temporarily for nonconforming works until project and construction activities meet the contract requirements.
4. Certify any cost changes due to labor, materials, and other items caused by the enacted government laws, statutes, and regulations. After consulting with the contractor, recalculating the new cost index and having the owner's approval, recalculate the new contract cost and/or adjust cost index.
5. In the construction bidding period, assist the owner to select the best contract-awarding format benefitting cost control and carefully review provisions related to cost control in the bidding documents.
6. In the construction preparation period, assist the owner to review the contractor's construction planning, scheduling, and cash flow plans, as well as the annual plan, and sign and issue advance payment notice.
7. During the construction period, verify the quantities of finished portions of the project, and sign and issue the notice of payment. Review quantity changes induced by project amendments, audit contractor proposed delivery extension or cost claim requests, and submit to the owner for approval. If necessary and appropriate, recommend counterclaim for loss to the owner for the contractor's liability.
8. In the completion acceptance and warranty period, assist the owner to prepare the final payment.

8.7.3 Methods and Procedures

Organizational measures:

- Establish reasonable, effective, and responsible organizations or teams.
- Develop a reasonable cost control workflow chart.

Economic measures:

- Decompose the project cost into reasonable blocks and levels.
- Analyze and predict various risks influencing project cost control.

- Develop a complete control system, emphasizing quantity measurement and payment management.
- Prepare a detailed project cost plan.

Technical measures:

- Compare technical and economic impacts for various engineering technologies and schemes.
- Compare technical and economic impacts for different construction technical schemes.
- Compare technical and economic impacts for contract changes during implementation.

Contract measures:

- Analyze and compare different contract awarding models, and select the best contract awarding model to control or reduce project cost.
- Carefully review cost-related provisions in the contract documents.
- Assist the contractor to take effective measures to reduce the project cost during contract excitation.
- Strictly control the contract or engineering change approval process, and control its influence ranges and reduce contract change-induced costs.
- Strengthen contract supervision, and properly manage claims.

8.7.4 Contingency Cost Control

During the execution of a construction contract, many situations may occur that involve the project contingency cost. A contingency cost claim is usually due to contract or engineering changes. Typical contingency costs and points for cost control are discussed as follows:

Contract change-induced contingency costs:

- When a contract change independent of the contractor exceeds the original design standards and scale, it must be submitted to the owner' design and planning departments for approval. Additional contract cost shall be based on the owner-approved cost change estimates.
- Carefully review the drawings, engineering estimates, and design documents. For costs induced by drawing errors, request the owner's design department or the contractor to complete contract or engineering change procedures as soon as possible, adjust engineering quantities and estimates, and revise the project cost accordingly.
- For contract changes proposed by the contractor, SE shall obtain the necessary information including fabricator's records, perform analysis and preliminary evaluation, determine the liability and its ranges, verify whether the contractor has emergency prevention and response plans, and provide recommendations for the owner's approval of the contract change procedure. Review the contractor's change order, and provide recommendations for the owner's approval. Cost shall be adjusted as per the owner's approval.

Claim-induced contingency costs:

- Material and labor cost adjustment claims are strictly based on contract documents. SE shall carefully examine supporting data and calculations.
- Payment delay claims should be implemented after the owner's approval. SE shall monitor current payment status and coordinate invoices and payments to avoid any potential payment delay claim.
- When predicting potential natural disaster losses, SE should bring the contractor's attention to timely enact the emergency response plan to minimize the disaster risk of damage. He or she must collect relevant evidences including photographs, records, and test data to provide basic information for potential claims.

- SE shall supervise the contractor to execute the contract according to the approved project schedule, plan of construction to organization, and coordinate stakeholders of the project to avoid labor- and equipment-wasting activities. In case of a contractor-induced project suspension and termination, SE shall assist the owner and the contractor to develop practical resolutions that will make project delay time and losses minimum. SE should also remind the contractor to collect claim evidences.
- Review after the requested claim is in complete compliance with the requirements of the contract documents and supervision specifications. SE shall review basic supporting materials carefully, check claim calculations, and provide recommendations for the owner's approval. Claim will be paid per the owner's approval.

8.8 Contract Management

Engineering project contract management and administration is a systematic and effective management process of contract creation, execution, and analysis to maximize financial and operational performance and minimize risk. It is to monitor and check the signing, implementing, changing, and terminating of a contract and resolve contract disputes and arguments based on laws and legal requirements. The main task of contract management is to ensure the contractor's compliance with the terms and conditions specified in the contract documents by directing, organizing, and supervising based on related laws and policies.

8.8.1 Contract Document Management

Project contract documents mainly include the following:

- Executed agreements
- Winning bid notice
- Bidding documents including addendum, supplementary materials, and clarification of memorandum of requests for information during contract negotiation
- Contract conditions
- Technical documents including standard construction specifications and special project provisions
- Plans and drawings indicating place, dimensions, materials, and written notes
- Verified engineering estimates quotations
- Other documents specific to the contract

All the aforementioned documents are managed by specific staff. Any use of the construction documents shall be kept confidential and recorded. The documents shall be returned immediately after use.

8.8.2 Construction Contract Management

1. Construction contract management is the active management of the relationship between the owner and the contractor over the term of a contract, mainly including supervising the full implementation of the construction contract, facilitating and negotiating contract changes, mediating contract disputes, and handling contract claims.
2. Notify and recommend the contractor in writing to replace unqualified or incompetent staffs or teams when quality and delivery schedules cannot be met due to incompetent management and staffing.
3. Dynamically manage the contract, and timely and promptly discover and correct any violation of the contract.

8.8.3 Subcontract Management

A bridge construction contract usually consists of several subcontracts for specific tasks and works. Due to the absence of a direct contractual relationship with the subcontractor, CSE must rely on the prime contractor to manage subcontract work. Since problems at the subcontract level can have a significant impact on the prime contractor's ability to meet his or her contractual obligations, SE must ensure that the prime contractor exercises adequate control over the subcontractors. The main responsibilities of SE are to review and verify the following items:

1. Conformance of the subcontract project's category and quantity with the general contract
2. Subcontractor's qualification including permits, manager's credentials, and construction equipment status
3. Technical specifications and acceptance standards adopted in the subcontract project
4. Conformance of the subcontract delivery schedule with the general contract.

8.8.4 Contract Change Order Management

1. A contract change order (CCO) is work that is added to or deleted from the original scope of work of a contract. It may involve changes of appearance, aesthetics, quantity, quality, specifications, and contractual requirements.
2. When a contract change exceeds the original design standard and/or scale, the complete change order including drawings and corresponding provisions should be submitted to the owner's original design and planning departments for approval.
3. When a contract change proposed by the contractor exceeds the original design standard and/or scale, the CCO request should be submitted to CSE and the owner for approval. When a contract change is proposed by SE or the owner, SE shall negotiate with the contractor and then issue the CCO.
4. The main procedures are as follows:
    a. For a CCO proposed by the owner or its designer, SE shall handle necessary documentary works per contract requirements.
    b. For a CCO involving only the process proposed by the contractor, SE shall perform a thorough review.
    c. For a CCO involving engineering issues proposed by the contractor, SE shall perform a preliminary review on the quantity changes due to the CCO and report to the owner's approval. The project shall be redesigned by the owner or designated designer.
5. SE shall collect and archive the CCO documents including the CCO, item quantity list, design drawing, letters, and requirements.

8.8.5 Claim Management

Common project claims are related to construction delays and cost. Other common claims are related to bid errors, late payment, delivery extension, disaster loss, defective work, liquidated damages, cost adjustments, contract termination, and fraud. For a delay-related claim, the contractor shall submit the extension request to SE for review and the owner for approval. For cost-related claims, SE shall investigate the supporting information of the claim and report to the owner for approval.

Typical claim management procedures are as follows:

- File a claims report and supporting documents.
- Investigate and verify basis, relevant facts, cost, data, and supporting documents of the claim with the contractor's concurrence.
- Review the rationality of relevant provisions of the contract documents used in the claim.

- Analyze and check the accuracy of the claim.
- Develop preliminary recommendations.
- Inform the preliminary recommendations to the contractor.
- Submit the preliminary recommendations to the owner for approval.
- Solve and handle the claim per the owner's approval. Develop and archive the final claim resolution report.

8.8.6 Construction Insurance Management

Construction insurance policies are usually specified in the relevant specifications. The contractor shall submit required insurance documents at the time the contract is executed. SE shall check and verify that the contractor carries the general liability and umbrella liability insurance covering all operations by or on behalf of the contractor, providing required insurance coverage for bodily injury liability, property damage liability, and third-party liability during the execution of the contract.

8.9 Construction Environmental Management

8.9.1 Main Tasks

The main tasks of an SE are as follows:

1. Monitor and ensure the contractor's conformance with government environmental protection requirements and standards for wastewater and solid waste treatment and discharge, air and noise pollution control, and so on.
2. Monitor and ensure normal operation of environmental protection facilities and implementation of environmental protection measures.
3. Periodically check conformance with national standards for drinking water, industrial water, and wastewater discharge.
4. Strictly control air quality and make sure that the fence goes all the way around the storage yard where the dustproof devices are installed.
5. Household sewage shall be filtered before entering the sewer pipe. Machinery- and equipment-washing sewage shall be treated by an oil/sand interceptor before being discharged. Construction sewage shall be treated by the sand interceptor before being discharged into drainage pipes.
6. Strictly control noise pollution in construction sites and neighborhood communities. Properly arrange the operation schedule, and limit the use of horns in transportation ship and vehicles.
7. Properly collect garbage, solid waste, and recyclable items.
8. Monitor all pollution sources and protection methods for x-rays, noise, dust, wastewater, solid waste, and so on.
9. Inspect the storage, use, and recycling of epoxy powder and spray painting dust.

8.9.2 Methods and Procedures

1. Urge the contractor to have a special environmental protection department and develop an environmental protection control and management system; SE must supervise both.
2. Designate special personnel to inspect environmental protection work.
3. Monitor and inspect the implementation of the environmental management plan, including basis for the plan, main pollution sources, discharge procedures, environmental standards, environmental protection facilities and processes, ecological prevention measures, and environmental protection cost estimation.

4. Inspect the operating conditions, effects, and efficiency of environmental protection facilities.
5. Establish an environmental protection meeting system. Conduct routine environmental protection working meetings and coordinate the related works.

8.10 Organizational Coordination

Many organizations, including the owner, designers, surveyors, consultants, general contractors, subcontractors, and supervision services, are involved in a bridge construction project. It is one of the most critical supervision tasks to coordinate all organizations into an effective and unified management pattern and standard system for a bridge construction project. Figure 8.2 shows the relationship between various teams for a bridge construction project.

8.10.1 Main Tasks

Organizational coordination, one of the most important tasks of supervision services, is usually specified in contract documents. The purpose of organizational coordination is to bring all participating organizations of a complex construction project into a harmonious or efficient relationship to achieve contract requirements. The main tasks of an SE are as follows:

1. Properly coordinate various interpersonal relationships, such as those between the CSE office and various SE teams and between CSE and the contractor, the owner, the designer, and other construction teams. Mainly solve project-related interpersonal conflicts.
2. Clearly determine and coordinate job assignments, duties, and responsibility correspondences and communications.
3. Effectively coordinate and balance supply and demand relationships, including man power, funds, equipment, techniques, and information services.

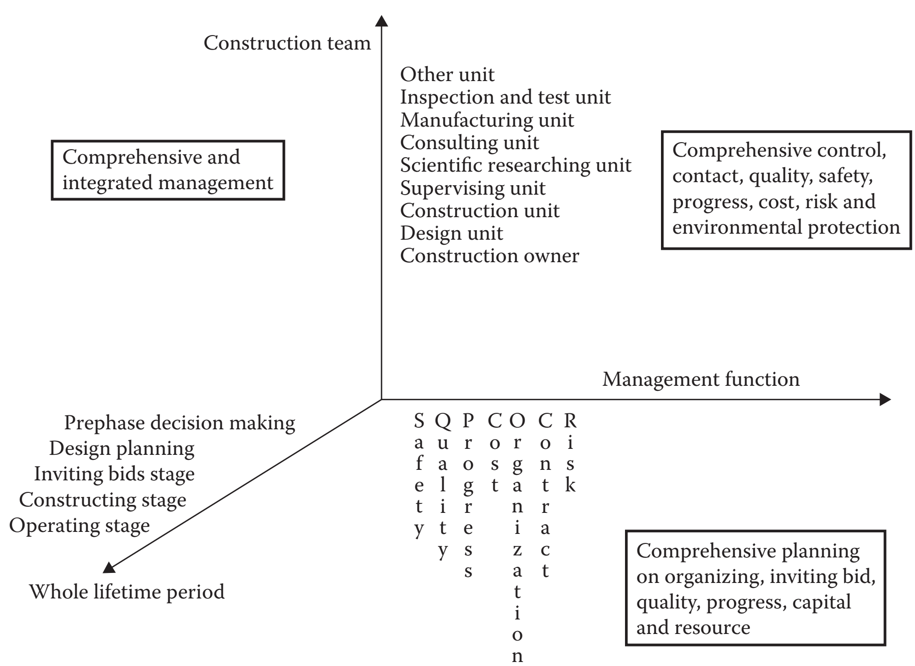

FIGURE 8.2 Bridge project coordination system plan.

4. Develop and coordinate cooperative relationships among all players involved in the project, including the owner, the contractor, subcontractors, suppliers, and supervision services, to work for the project's goals.
5. Fully understand governmental laws, regulations, polices, and law enforcement procedures and details to help and coordinate various communications between the contractor and traffic operation, road control, environmental agencies, and utility companies.
6. Develop symmetrical, workable, and effective organization charts and operation procedures for the project.

8.10.2 Methods and Procedures

On-site meeting is an important form of organizational coordination. There are three types of on-site meetings: (1) first on-site meeting, (2) regular on-site meeting, and (3) special on-site meeting.

8.10.2.1 First On-Site Meeting

The first on-site meeting with the owner, the contractor, and all related parties is facilitated by the SE. Main agendas of the meeting are to review and check every preparation work prior to construction and clarify supervision procedures; the following items are usually discussed:

- QA and QC procedures
- Project delivery processes
- Invoice and payment procedures
- Delay and claim procedures
- CCO procedures
- Quality and safety incidents reporting procedures
- Correspondence procedure
- Regular meeting schedule

8.10.2.2 Regular On-Site Meeting

A regular on-site meeting is usually scheduled once per month. Agenda items are as follows:

- Examine project delivery progress, and identify causes of factors affecting progress and find out solutions.
- Examine the supply–demand circumstance of the construction site including man power, equipment, and materials.
- Examine quality of the project and provide QA inspection report and recommendations to improve quality deficiency and control procedures and processes.
- Examine invoice and payment issues.
- Examine all construction safety issues to avoid potential hazards.
- Discuss the construction environment.
- Clarify potential delays and claims.
- Other issues.

8.10.2.3 Special On-Site Meeting

If necessary, a special on-site meeting is held to discuss work arrangements, important safety and quality issues, problems raised from the construction site, and so on. The participants are supervision staff including CSE and directors of all SE groups, contractor's project manager and chief project engineer, subcontractor's project manager, and the owner.

8.11 Construction Safety

One of the primary responsibilities of an SE is to ensure that a safe working environment and safe working practices as specified in the contract specifications are maintained at the project site. This may involve monitoring and checking the operation of equipment, use of personal safety equipment, and installation of warning signs.

Contractors should set an example by following the code of safe practice and also by using personal safety equipment including hard hats, gloves, and protective clothing. In addition, they must enforce the safety guidelines specified in contract specifications.

8.11.1 Main Tasks

1. Randomly inspect safety facilities including appliances, equipment, and major equipment maintenance and operation records.
2. Monitor safety techniques used in construction.
3. Provide necessary technique consultation for typical safety problems.
4. Inspect all on-site operations to avoid any potential hazards.

8.11.2 Methods and Procedures

1. Review and confirm the related documents, reports, and forms:
   a. Verify safety qualifications and certificates of all subcontractors entering the construction site, the contractor's safety coordinators, safety training certificates, and certificates for special operations.
   b. Review the contractor's construction plans.
   c. Review the contractor's safety reports and statistical data.
   d. Review project safety accident report.
   e. Review subproject safety inspection report.
   f. Review and sign safety technique documents, and so on.
2. Supervise and urge the contract unit (contractor) to do safety inspection:
   a. The contractor must designate an on-site safety coordinator during ongoing operations on the jobsite all the time.
   b. The contractor's safety coordinator must check all safety issues for every process and every stage and correct undesirable unsafe operations in time.
   c. The contractor's safety coordinator must follow approved safety inspection plans and procedures and submit complete, accurate, reliable, and true data and materials.
   d. An SE may certify the data and materials provided by the contractor's safety coordinator.
3. On-site monitoring:
   a. Before starting work, ensure that contractors (1) submit all required safety policies, plans, and project-specific safety practices; (2) present a safety training program; (3) submit permits required before starting certain work, such as excavation, trenching, shoring, falsework erection, and scaffolding.
   b. Perform daily routine inspection on the safety situation of main structures and components and random testing if necessary.
   c. Ensure the contractor complies with all aspects of the contract, including applicable safety orders, and participates in and organizes investigations on safety incidents.
4. Ensure construction operation compliance with the government's occupational safety and health standards.

5. Pay special attention to unsafe human behaviors and unsafe object conditions, and monitor safety and environment conditions in surrounding areas.
6. Strictly implement all safety standards, regulations, and procedures on a project site. Develop a detailed safety management and evaluation procedure, responsibilities, and obligations:
   a. The contractor shall collect weather forecast information and prepare for extreme weather including strong wind, heavy storm, hurricane, and high temperature.
   b. The contractor shall keep fire-extinguishing equipment in good condition and the fire path clear.
   c. The contractor shall check power supply lines periodically to ensure electrical safety.
7. The contractor shall conduct safety training programs and regular safety meetings for all employees.

8.11.3 Safety in Major Construction Operations

1. Excavation and trenching are among the most hazardous construction operations. Cave-ins pose the greatest risk, resulting in worker fatalities. Other potential hazards include explosions, falls, falling loads, hazardous atmospheres, and incidents involving mobile equipment from the operation.
2. Temporary structures including falsework, scaffolding, shoring, sloping, benching, guying systems, concrete forms, and heavy lighting equipment support various large loads during construction operation. Safety incidents frequently occur due to the collapse of temporary structures.
3. The prestressing operation can be potentially dangerous due to the tremendous forces involved. In case of a failure, high-velocity projectiles are produced. SE should always stay alert and be aware of the contractor's operations and equipment. Before the contractor begins the stressing operation, SE shall check all of the high-pressure hoses for leaks and/or poor condition and stay clear of the immediate operation area.
4. Operations in high places require special attention. High place bridge construction involving erection and assembly operations is of high risk.
5. Electricity has long been recognized as a serious workplace hazard, exposing employees to dangers such as electric shock, electrocution, fires, and explosions. All electrical operation should be in compliance with government's construction safety and health standards.
6. Overwater operations present all of the usual construction hazards plus additional hazards posed by the marine environment. Construction barges and ships must not be operated without government permits and certificates. SE should ensure that navigation, anchorage, mooring, and operation of all floating facilities are in compliance with national maritime safety provisions. Signs for navigation, rescue, and extinguishing equipment shall be installed in proper locations. A safety procedure shall be developed. The contractor shall monitor local meteorological and hydrological situations during construction to make sure the floating facilities anchored properly.

8.12 Documentation and Records

8.12.1 Information Management

Supervision information management includes internal computer-aided information management of documents, files, supervision reports, forms, construction site meetings, and so on and information service equipment such as remote video monitoring systems provided by the owner. The main tasks of information management may include the following:

1. Connect to the owner's information system in accordance with the owner's management protocol.
2. Set up two levels of information management: chief supervision office and supervision group. Designate specific information management staff; develop supervision information management plans according to contract documents, supervision specifications, and government requirements.
3. Collect statistical information related to the project accurately, and report to chief supervision office via the information management system.
4. Collect, reorganize, and analyze information data during construction to provide basis for project quality inspection, acceptance check, as well as future maintenance and management.

8.12.2 Documentation and Records

SE is also responsible for maintaining an accurate and complete record of work completed by the contractor. Supervision or project records and reports are necessary to determine that contract requirements have been met so that payments can be made to the contractor. The supervision records should be kept current, complete, and accurate and should be submitted on time. The supervision records include correspondences, diaries, weekly reports, monthly reports, special reports, meeting minutes, final reports, other original records, as-built plans, and final completion documents:

1. Diary: it is critical that the SE keeps a written diary of the activities that take place in the field. The diary should contain information concerning the work being inspected, including unusual incidents and important conversations. This information may become very critical in case of a legal action for litigation involving construction claims or job failure. The diary shall be submitted to the project team for filing after the project is completed.
2. Weekly report: it mainly summarizes weekly project progress and safety and quality condition in a specific format. The report is usually written by a field SE and submitted to CSE for approval every Sunday. The approved report will be submitted to the owner the following Monday.
3. Monthly report: it mainly summarizes monthly project process and safety and quality condition in a specific format. The report is usually written by the field SE and submitted to CSE for approval before the second day of each month. The approved report will be submitted to the owner before the fifth of each month.
4. Correspondence: written communications with the contractor, the fabricator, the owner, and other project stakeholders.
5. Special report: it mainly includes major quality and safety incidents, emergencies, events, or matters that the field SE deems necessary to report. It is written by the field SE and approved by CSE; the report will be submitted to the owner.
6. Meeting minutes: it summarizes the attendees, agenda, items, discussions, resolutions, and actions in a meeting.
7. Final report: it is written by a site SE when the project is completed and inspection results demonstrate that all contract requirements for the contractor have been satisfied with the final acceptance at the jobsite. The report shall be reviewed and approved by CSE and submitted to the owner.
8. Other original records: these include the quality inspection report of the contractor, various QA testing records of sampling inspections, and so on.
9. As-built plans: as-built plans, also referred to as as-constructed final plans or marked final plans, are original as-awarded project plan sheets that have been updated to show changes that occurred during construction. As-built plans should reflect any deviation that may exist between the as-awarded project plans and what were built in the field. Accurate and complete as-built plans are very important and useful for maintaining the constructed bridge and for any future work on the bridge. The as-built plans also provide input and information for future seismic retrofits of the bridge.

10. Final completion documents: after the project is completed, the supervision office shall submit all documents and records during the supervision process to the owner within the period specified in the contract. The main contents are supervision contracts, supervision implementation plans, correspondences, memorandums, notices, meeting minutes, weekly reports, monthly reports, original quality sampling inspection reports, quality incident reports, special reports, quality evaluation reports, final reports, as-built plans, other materials that the project owner deems necessary to file, and so on.

8.13 Steel Box Girder Fabrication Quality Assurance Inspection

Steel box girders are among the most complex fabricated structural elements for large bridge structures. This section discusses major QA issues and challenges during steel box girder fabrication.

8.13.1 Source Inspection

Source inspection (Caltrans 2012b) is an integral part of an effective QA acceptance program. Source inspection is the acceptance testing of manufactured and prefabricated materials at locations other than the jobsite. An SE may be required to conduct source inspection per contract requirements. Verification source inspection helps to ensure that structural materials incorporated into bridge structures comply with contract requirements with regard to raw materials, fabrication processes, personnel certifications, and in-process QC testing. The purposes of verification source inspection are to

- Verify that adequate off-site QC is in place.
- Perform verification sampling and testing of representative material.
- Perform necessary in-process verification inspections.
- Perform NDT at appropriate times.
- Mitigate issues before the material is shipped to the jobsite.
- Decrease the potential for project delays that verification sampling at the jobsite may cause.

8.13.2 First Segment Approval Practice

In the steel bridge fabrication industry in China, it is requited to approve the first segment prior to mass production. This practice is based on the principle of "prevention first and pilot project first" to comprehensively evaluate the quality indicators of the first segment and to develop a guide in mass production. Mass production shall not proceed without the approval of the first segments.

8.13.2.1 Main Tasks

The main tasks of an SE are as follows:

1. Review and approve the implementation plan and work instruction of first segment submitted by the contractor or the fabricator; develop the corresponding supervision plan.
2. Verify the use of proper materials by checking mill test reports (MTRs) when the steel plates arrive and by monitoring heat numbers during fabrication until the material is joined into a piece-marked item. Periodically verify the following information on MTRs: product description (specifications, grade, and H or P testing frequency); chemistry; physical test results, including Charpy V-notch when applicable; heat number; and certification signature (quality control department and notary, when required).
3. Continuously monitor the whole fabrication process of the first segment on site; ensure the proper implementation of the approved process, and record all details. Bring any problems to the attention of the fabricator immediately upon discovery. Corrective actions shall be taken upon consensus between the fabricator, SE, and owner to ensure smooth implementation.

HSE inspection of the first segment is as follows:

1. After the completion of the first segment of steel box girder fabrication and preassembling, inspection, acceptance, and assessment should be performed; a first segment on-site peer review meeting should be organized to verify the reliability and rationality of the fabrication processes and to write the assessment report and suggest improvements.
2. After the first segment passes the inspection and assessment, SE should require the fabricator to further improve the construction implementation plan according to the assessment report and thus improve implementation of the supervision program.
3. SE is responsible for organizing site meetings to promote the fabrication practice of the first segment to mass production with the same quality level. In this respect, SE must make greater efforts in the mass production process to ensure the following:
   a. Execution of the process program.
   b. Quality inspection and acceptance criteria will not be reduced.
   c. QC plans are implemented properly and practically.
   d. Construction personnel qualifications, equipment precision and adjustment, and the inspection capacity meet the requirements.

8.13.2.2 Implementation Plan

1. Preparation-stage requirements: before the fabrication of the first segment, the requirements for the preparation stage are as follows:
   a. SE shall require the contractor to carefully complete fabrication preparation and submit fabrication plan for approval as required.
   b. SE shall require the contractor to submit fabrication plan and QC plan and other technical documentations for the first segment 45 days in advance. SE shall review them in terms of adequacy and standard compliance to ensure that all fabrication activities comply with required regulations, requirements, and within a controlled condition. The submitted technical documentations include but are not limited to the following:
      i. Fabrication plan describing all fabrication programs, process regulations, on-site management requirements, equipment, and man power allocation.
      ii. Fabrication process, programs and other standard documents such as welding procedure qualification and test program and reports, blasting process, and transport programs.
      iii. The QC plan including procedures, standards, personnel, polices, and records used during planning, ordering, fabrications, inspection, cleaning, painting, and shipping of fabricated members and quality management and assessment methods and measures.
      iv. Quality risk assessment report, mainly involving major quality risks and appropriate preventive and response programs.
      v. HSE management program, effectively identifying various risk sources of safety, environmental protection, public health, and corresponding preventive and response alternatives.
2. Construction-stage requirements: before the fabrication of the first segment, the requirements for the construction are as follows:
   a. SE shall supervise or participate in the contractor's training programs for the management team and technical staff, including welders, painters, and operators, to ensure that all participants fully understand their roles and responsibilities, process control standards, process management and coordination, documents management, and methods and measures to deal with problems and related assessment and constraint mechanisms.
   b. SE shall participate in the contractor's quality management and control training for its management team. Welders, painters, lifting-equipment operators, and other specialized workers must be certified by approved professional agencies and institutes. The contractor should submit all certifications and qualifications for fabricator, QCI, welder/welding operator, and

lifting-equipment operator to SE for review and verification. Only certified or qualified personnel are authorized to work in their specified work.

3. Fabrication work order: before the fabrication of the first segment, SE should check all contractor's preparation works for both preparation and construction stages.

The SE shall issue the fabrication work order only when all the preparation works as discussed previously in this section are satisfied.

8.13.2.3 Fabrication Summary and Acceptance Review

1. After the completion of the first segment, the contractor shall submit the first segment fabrication report to the SE and the owner for review. The report should contain at least the following main elements:
   a. Construction situations and important construction management activities.
   b. Construction organization, implementation of the construction process, and contract changes (if any).
   c. QC inspection and mitigation (if any).
   d. Construction management evaluation and summary.
   e. The main methods and measures for improvement.
   f. List of construction quality.
   g. Construction quality mitigation list.
   h. Construction management and quality inspection forms and data.
   i. A complete set of fabrication processes, techniques, management measures, quality standards, inspection methods, and evaluation forms recommended for mass construction.
2. The contractor shall coordinate with the SE to organize a technical summary and evaluation meeting for first segment fabrication; revise fabrication processes, techniques, management measures, quality standards, inspection methods, and evaluation forms recommended for mass construction based on the meeting's recommendations; and then submit revised ones to the SE for approval.
3. Mass construction is not permitted without the SE's approval.

8.13.3 Quality Assurance Testing and Inspection

When fabricating bridge steel structures, quality sampling and NDT and sampling of steel plate raw materials, welding materials, and welding joints are usually conducted by an SE or by designated testing supervising units. The chief supervision office should establish a testing department with professional testing engineers in charge of programming, planning, sampling, delivering, observing testing, preparing testing summary reports, and so on.

8.13.3.1 Inspection Scope and Items

1. Raw materials: steel plates, welding metals, high-strength bolts, and so on
2. Physical quality: field and shop welded or bolted joints and connections
3. Test and examination: welding test piece, welding procedure evaluation and welder qualification test, and so on

8.13.3.2 Testing Items and Inspection Percentages

The sampling percentages for the following inspection items are as follows:

1. Weld joint appearance inspection: weld size, profile, and contour of fillet and groove welds: 20% of the contractor's QC inspection
2. NDT of a weld joint: radiographic, ultrasonic, and magnetic particle testing: 20% of the contractor's QC inspection
3. Physical and chemical testing of raw materials: 20% of the contractor's QC inspection
4. Coating thickness and adhesion inspection: 20% of the contractor's QC inspection

5. High-strength bolts and shear studs: 20% of the contractor's QC inspection
6. Welding rest piece: 20% inspection, and provide testing report including weld penetration of acid etching test for U-rib test plate weld joint slice
7. Welder/welding operator qualification test
8. Welding procedure qualification test: ultrasonic testing for welding procedure for bridge deck U-rib weld joint

8.13.3.3 Selection of Welding Spots for Inspection

1. Key spots and parts with more quality problems should be sampled more frequently than other parts. For instance, steel anchor boxes, steel box girder anchor box joints, transverse weld joints, and other main stressing areas should be sampled specifically.
2. Both key parts and general parts should be sampled randomly.
3. Proportion of sampling should be increased during construction period and quality unstable period.

8.13.3.4 Testing Procedures

On the basis of contractor's QC inspection, SE should determine the testing locations, details, and requirements and inform each branch of the testing department in the written form of sampling requisition; each branch should perform testing according to the sampling requisition and submit the inspection result report to SE for review and signing. SE may provide some recommendations for discovered excessive defects.

8.13.3.5 Testing Quality Requirements

1. The contractor shall submit QC inspection reports or certificates for raw materials and equipment for fabrication. The testing department should recheck their conformance with applicable specifications covering common requirements for hot-rolled plates, shapes, sheet pilling, and bars after obtaining the contractor's QC inspection reports.
2. Testing SE is responsible for sampling and specimen manufacturing. The whole sampling process should be performed randomly.
3. Before the testing samples and specimens are delivered to the testing laboratory, the testing SE should number and register all samples and specimens with detailed information such as the contractor's name, material batch or component location, and sampling or specimen manufacturing location.
4. Testing SE should monitor and supervise the sample delivery without any changes, damage, or contamination.
5. The testing department should be responsible for the authenticity and representativeness of the samples delivered, and the testing laboratory should be responsible for the testing results.
6. The testing laboratory should perform sample testing and assessments according to the specified test specifications and is responsible for the testing results of weld joints. Test records shall be clear, accurate, and complete. Detected defects shall be marked clearly and recorded in the inspection result report and submitted to the SE.
7. The chief supervision office shall review the certifications of testing personnel, check testing facilities, and ensure the testing department's compliance with the contract documents and test specifications.
8. The testing department should perform the tests in accordance with the department's management system to achieve standard management, operation, recording, and high quality.
9. The testing department should carefully maintain and calibrate the testing instruments and equipment, to ensure that the instruments and equipment are of good condition and required accuracy.
10. The testing department should develop internal training and performance management procedures; timely replace unqualified testing staff; and determinedly dismiss those who are irresponsible, are fraud, or abuse power.

8.14 Summary

This chapter discusses the job content of construction supervision related to large bridges and its procedures and methods of principle. Meanwhile, combined with the present development trend of bridge engineering, it deeply analyzes and discusses the key points and difficulties of steel structure building supervision of large bridges and targeted supervision measures. It is believed that such procedures, methods of principle, or targeted measures are applicable to the construction supervision of small and medium bridges also.

References

AASHTO. 2011. *AASHTO LRFD Bridge Construction Specifications*, 3rd Edition, with 2010 and 2011 Interim Revisions, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO/AWS 2010. *Bridge Welding Code*, D1.5M/D1.5:2010, 6th Edition, American Welding Society, Miami, FL.

AASHTO/NSBA. 2002. *Steel Bridge Fabrication QC/QA Guide Specification*, S4.1, American Association of State Highway and Transportation Officials and National Steel Bridge Alliance Steel Bridge Collaboration, Washington, DC.

Caltrans. 2010. *Standard Specifications*, California Department of Transportation, Sacramento, CA.

Caltrans. 2012a. *Construction Manual*, California Department of Transportation, Sacramento, CA.

Caltrans. 2012b. *Source Inspection Quality Management Plan (SIQMP) Outline*, California Department of Transportation, Sacramento, CA.

MOTPRC. 2011. *Highway Bridge and Tunnel Construction Specifications*, JTG/T F50-2011, Ministry of Transport of the People's Republic of China, Beijing, China. (In Chinese)

Re, R. D. and McKittrick, H. V. 1985. "The Role of the Resident Engineer," *Proceedings of the Specialty Conference*, Tampa, FL, Apr. 15–17, ASCE, New York, NY.

9

Bridge Management Using Pontis and Improved Concepts

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9.1 Introduction

Bridge management is an important activity of highway transportation agencies. The practice of bridge management in the United States is at a stage of combining traditional approaches with computerized approaches such as quantitative analysis through computer software programs referred to as bridge management systems (BMS). The development of BMS in the United States has advanced over a period of about two to three decades.

The traditional approaches and computerized approaches to bridge management currently complement each other. The former has the advantage of being able to take into account factors not easily quantifiable, such as the significance of historical value of a bridge and political pressure from the legislature branch of the government regarding a route. The latter offers a solid quantitative analysis and is able to simultaneously include and cover a large amount of factors. Their combination appears to be advantageous in dealing with real-life situations where a large variety of factors need to be taken into account.

There have been a number of efforts in the bridge engineering and transportation engineering community on developing suitable BMS. In the United States, besides the major efforts started in the 1990s on Pontis and BRIDGIT, individual state departments of transportation also have invested human and monetary resources in developing BMS within their own jurisdiction. Having witnessed staged successful developments of Pontis, many efforts on other bridge systems to an operational stage have virtually stopped or suspended in the United States with a few completed ending with systems not as comprehensive as Pontis (AASHTO 2005). Pontis has gained an overwhelming momentum to be dominant in this market, with a number of states changing their directions of effort to adopting Pontis. It appears that

Pontis will remain dominant in the U.S. practice of bridge management for years to come. Note also that a number of other countries have adopted the concepts used in Pontis in developing their BMS. As a result, this chapter will have a focus on Pontis and its possible improvement.

An important goal of BMS is to minimize the life cycle cost for individual bridges and for entire network of bridges with the jurisdiction. Nevertheless, simultaneously reaching both optima may become too costly or not possible due to constraints of available computing resources. Including all details of each and every bridge in the network will require advanced computing power to process the information. Such computing power is often not easily available for state agencies in the United States. Accordingly, the goals of bridge-based (or project-based) and network-based optimization are reached separately. The network-level optimization is done considering only limited information on each bridge's components. Then prioritized bridges (or projects) can be further analyzed with more details than those used in network-level optimization.

In these two processes of optimization, a critical step is to reliably predict condition deterioration or improvement of the bridge components within the jurisdiction. This is done using previous inspection data as the history of condition evolution. Appropriate or optimal actions can then be taken to minimize the deterioration rate (life cycle cost) and maximize the effect of spending for replacement or maintenance, repair, and rehabilitation. In the United States, Pontis is now the most popular BMS among the state transportation agencies, while other traditional approaches are still being used for decision making. The deterioration model in Pontis for bridge elements is based on the Markov Chain theory, with a statistical regression for estimating the required transition probability matrix. This is the core part of deterioration prediction in Pontis.

Markov Chain is a model for stochastic processes whose condition or state evolves with time. There is an extensive literature for its application to a wide range of practical problems (e.g., Fu and Moses 1986; Fu 1987; Puterman 1994; Baik et al. 2006). The critical part of a Markov Chain is its transition probability matrix that models the evolution process, with a measurement for the likelihood of the system to evolve from one condition to another. This likelihood is quantified based on the theory of probability. For practical application, estimation of these transition (evolution) probabilities is therefore critical for the reliability of modeling.

9.2 Pontis Method of Transition Probability Estimation for Modeling Element Deterioration

Pontis updates the transition probabilities using two sources. One is expert elicitation and the other historical inspection data. The expert elicitation is simply input by the user, which can be based on experience without inspection data at all. At current stage of Pontis application, most of experience perhaps has to be derived from inspection data. Thus, the focus here is on how to use historical inspection data to estimate or update the transition probabilities (particularly for the "do-nothing" option) to model a period of time where no maintenance or replacement is done until a serious issue is brought to the attention.

To determine the transition probability matrix for a bridge element in an environment in the jurisdiction of an agency, two phases of calculations are used in Pontis. The first one is to estimate such matrices using inspection data according to their inspection intervals. The second one is to combine these matrices into one. The need for the first phase is due to the reality that not all bridges are inspected with a constant time interval.

9.2.1 Estimation of Transition Probabilities for One-Time Step Using Inspection Data

Estimating the transition probabilities in Pontis for modeling deterioration for one-time step is proceeded as follows: (1) Identifying pairs of the "before" (at time t_{n-1}) and the "after" (at time t_n) condition data. (2) Using the identified paired data to compute or estimate the transition probability matrix by regression. Note that the time step $t_n - t_{n-1}$ here can be 1, 2, 3 years, and so on. The first step of identifying data pairs is to prepare relevant data for the second step of computation-based estimation. It includes assembling pairs of condition inspection data over time for the specific element and making sure of consistent time intervals between inspections.

For each observation pair of inspection data, vector ***hj*** is used to record the pair (Pontis Technical Manual 4.4, 2004):

$$\boldsymbol{hj} = \left\{x_1^j, x_2^j, x_3^j, x_4^j, x_5^j;\ y_1^j, y_2^j, y_3^j, y_4^j, y_5^j\right\} \tag{9.1}$$

where x_k^j is the bridge element in condition state k that has been observed in the earlier ("before") observation of pair j, and y_k^j is the element quantity observed in the kth condition state in the later ("after") observation for the same bridge. Hence, x_k^j and y_k^j (k = 1, 2, 3, 4, 5) form the pair.

For a bridge network and perhaps also a specific environment, there could be M such observation pairs for a specific element. This results in the following vectors $\boldsymbol{X}$ and $\boldsymbol{Y}$:

$$\begin{aligned}\boldsymbol{X} &= (x_1, x_2, x_3, x_4, x_5) \\ &= \left(\sum_{j=1}^{M} x_1^j, \sum_{j=1}^{M} x_2^j, \sum_{j=1}^{M} x_3^j, \sum_{j=1}^{M} x_4^j, \sum_{j=1}^{M} x_5^j\right)\end{aligned} \tag{9.2}$$

$$\begin{aligned}\boldsymbol{Y} &= (y_1, y_2, y_3, y_4, y_5) \\ &= \left(\sum_{j=1}^{M} y_1^j, \sum_{j=1}^{M} y_2^j, \sum_{j=1}^{M} y_3^j, \sum_{j=1}^{M} y_4^j, \sum_{j=1}^{M} y_5^j\right)\end{aligned} \tag{9.3}$$

Note that after the summation, these two vectors can be divided by the total quantity $\sum_{j=1}^{M} x_1^j + \sum_{j=1}^{M} x_2^j + \sum_{j=1}^{M} x_3^j + \sum_{j=1}^{M} x_4^j + \sum_{j=1}^{M} x_5^j = \sum_{j=1}^{M} y_1^j + \sum_{j=1}^{M} y_2^j + \sum_{j=1}^{M} y_3^j + \sum_{j=1}^{M} y_4^j + \sum_{j=1}^{M} y_5^j$ to express them in percentage or probability. Pontis then uses these vectors to estimate the transition probabilities through a regression procedure as follows.

Based on the total probability theorem, transition probabilities p_{ki} (i = 1, 2, 3, 4, 5) need to satisfy the following equation according to the total probability theorem:

$$y_i = p_{1i}x_1 + p_{2i}x_2 + p_{3i}x_3 + p_{4i}x_4 + p_{5i}x_5 \quad (i = 1, 2, 3, 4, 5) \tag{9.4}$$

Note that there are five such equations in Pontis for i = 1, 2, 3, 4, 5 to include all 25 transition probabilities in the matrix defined in Equation 9.4.

Due to random behavior of deterioration and possible variation in inspection data y_i and x_i (i = 1, 2, 3, 4, 5), Equation 9.4 cannot be satisfied exactly. In estimating the transition probabilities p_{ij}, Pontis uses the concept of regression, although there can be other approaches to finding them. Namely, Pontis finds such $p_{ij}(i, j = 1,2,3,4,5)$ values that minimize the differences between the two sides of Equation 9.4. Namely, the two sides of Equation 9.4 are not equal but their difference is minimized. This is the difference between the predicted and the observed conditions. This difference is defined as the sum of the squared residuals as follows:

$$\Delta_i^2 = \sum_{j=1}^{M} (y_i{}^j - p_{1i}x_1{}^j - p_{2i}x_2{}^j - p_{3i}x_3{}^j - p_{4i}x_4{}^j - p_{5i}x_5{}^j)^2 \quad i = 1, 2, 3, 4, 5 \tag{9.5}$$

To minimize Δ_i^2, differentiating this quantity with respect to $p_{1i}, p_{2i}, p_{3i}, p_{4i}$, and p_{5i}, and then equating the partial derivatives to zero, we obtain the following five linear equations as a set:

$$
\begin{aligned}
\sum_{j=1}^{M} x_1^j y_i &= p_{1i}\sum_{j=1}^{M}(x_1^j)^2 + p_{2i}\sum_{j=1}^{M}x_1^j x_2^j + p_{3i}\sum_{j=1}^{M}x_1^j x_3^j + p_{4i}\sum_{j=1}^{M}x_1^j x_4^j + p_{5i}\sum_{j=1}^{M}x_1^j x_5^j \\
\sum_{j=1}^{M} x_2^j y_i &= p_{1i}\sum_{j=1}^{M}x_2^j x_1^j + p_{2i}\sum_{j=1}^{M}(x_2^j)^2 + p_{3i}\sum_{j=1}^{M}x_2^j x_3^j + p_{4i}\sum_{j=1}^{M}x_2^j x_4^j + p_{5i}\sum_{j=1}^{M}x_2^j x_5^j \\
\sum_{j=1}^{M} x_3 y_i &= p_{1i}\sum_{j=1}^{M}x_3^j x_1^j + p_{2i}\sum_{j=1}^{M}x_3^j x_2^j + p_{3i}\sum_{j=1}^{M}(x_3^j)^2 + p_{4i}\sum_{j=1}^{M}x_3^j x_4^j + p_{5i}\sum_{j=1}^{M}x_3^j x_5^j \qquad \text{for } i = 1,2,3,4,5 \\
\sum_{j=1}^{M} x_4^j y_i &= p_{1i}\sum_{j=1}^{M}x_4^j x_1^j + p_{2i}\sum_{j=1}^{M}x_4^j x_2^j + p_{3i}\sum_{j=1}^{M}x_4^j x_3^j + p_{4i}\sum_{j=1}^{M}(x_4^j)^2 + p_{5i}\sum_{j=1}^{M}x_4^j x_5^j \\
\sum_{j=1}^{M} x_5 y_i &= p_{1i}\sum_{j=1}^{M}x_5^j x_1^j + p_{2i}\sum_{j=1}^{M}x_5^j x_2^j + p_{3i}\sum_{j=1}^{M}x_5^j x_3^j + p_{4i}\sum_{j=1}^{M}x_5^j x_4^j + p_{5i}\sum_{j=1}^{M}(x_5^j)^2
\end{aligned}
\tag{9.6}
$$

All these linear equations can be expressed in the matrix form as follows:

$$
[XX] = \begin{pmatrix}
\sum_{j=1}^{M}(x_1^j)^2 & \sum_{j=1}^{M}x_1^j x_2^j & \sum_{j=1}^{M}x_1^j x_3^j & \sum_{j=1}^{M}x_1^j x_4^j & \sum_{j=1}^{M}x_1^j x_5^j \\
\sum_{j=1}^{M}x_2^j x_1^j & \sum_{j=1}^{M}(x_2^j)^2 & \sum_{j=1}^{M}x_2^j x_3^j & \sum_{j=1}^{M}x_2^j x_4^j & \sum_{j=1}^{M}x_2^j x_5^j \\
\sum_{j=1}^{M}x_3^j x_1^j & \sum_{j=1}^{M}x_3^j x_2^j & \sum_{j=1}^{M}(x_3^j)^2 & \sum_{j=1}^{M}x_3^j x_4^j & \sum_{j=1}^{M}x_3^j x_5^j \\
\sum_{j=1}^{M}x_4^j x_1^j & \sum_{j=1}^{M}x_4^j x_2^j & \sum_{j=1}^{M}x_4^j x_3^j & \sum_{j=1}^{M}(x_4^j)^2 & \sum_{j=1}^{M}x_4^j x_5^j \\
\sum_{j=1}^{M}x_5^j x_1^j & \sum_{j=1}^{M}x_5^j x_2^j & \sum_{j=1}^{M}x_5^j x_3^j & \sum_{j=1}^{M}x_5^j x_4^j & \sum_{j=1}^{M}(x_5^j)^2
\end{pmatrix}
\tag{9.7}
$$

$$
[XY]_i = \begin{pmatrix}
\sum_{j=1}^{M}x_1^j y_i^j \\
\sum_{j=1}^{M}x_2^j y_i^j \\
\sum_{j=1}^{M}x_3^j y_i^j \\
\sum_{j=1}^{M}x_4^j y_i^j \\
\sum_{j=1}^{M}x_5^j y_i^j
\end{pmatrix} \qquad i = 1, 2, 3, 4, 5
\tag{9.8}
$$

$$a_i = \begin{pmatrix} P_{1i} \\ P_{2i} \\ P_{3i} \\ P_{4i} \\ P_{5i} \end{pmatrix} \quad i = 1, 2, 3, 4, 5 \tag{9.9}$$

The solution in Equation 9.9 can be written as

$$a_i = [XX]^{-1}[XY]_i \quad i = 1, 2, 3, 4, 5 \tag{9.10}$$

where superscript "–1" means inverse of matrix.

For the solution to exist, the matrix $[XX]$ must be nonsingular and thus invertible. Note that vectors $[\boldsymbol{XY}]_i$ $(i = 1, 2, \ldots, 5)$ can be assembled to one matrix $[XY]$ as follows:

$$[XY] = \left[[XY]_1, [XY]_2, [XY]_3, [XY]_4, [XY]_5\right] \tag{9.11}$$

Then the transition probability matrix **P** for one-time step can be written as follows according to Equation 9.9:

$$[a_1, a_2, a_3, a_4, a_5] = \mathbf{P}_e = [XX]^{-1}[XY] \tag{9.12}$$

This is the calculation in Pontis for estimating **P** over a time step or a time interval associated with the observation pair X and Y. The estimated matrix is now denoted as $\mathbf{P}_e$, with a subscript "e" for "estimated."

In practical application, $[XX]^{-1}$ does not always exist. In addition, for a typical interval $t_n - t_{n-1}$ of 1, 2, or 3 years, condition transition is not expected to "skip" a state and the condition is not expected to become better without maintenance, repair, or replacement work. Namely, p_{13}, p_{14}, p_{15}, p_{24}, p_{25}, p_{35}, p_{21}, p_{31}, p_{32}, p_{41}, p_{42}, p_{43}, p_{51}, p_{52}, p_{53}, and p_{54} are expected to be zero. Nevertheless, Equation 9.12 does not produce p_{13}, p_{14}, p_{15}, p_{24}, p_{25}, p_{35}, p_{21}, p_{31}, p_{32}, p_{41}, p_{42}, p_{43}, p_{51}, p_{52}, p_{53}, and $p_{54} = 0$ (Fu and Devaraj 2008). This is because the regression process does not require all these terms to be zero. In addition, each row also may not add to 1, again because it is not required in the regression approach used in Pontis.

However, after the calculation shown in Equation 9.12, Pontis takes only the diagonal terms for the transition probability matrix, sets a zero to p_{13}, p_{14}, p_{15}, p_{24}, p_{25}, p_{35}, p_{21}, p_{31}, p_{32}, p_{41}, p_{42}, p_{43}, p_{51}, p_{52}, p_{53}, p_{54}, compute $1 - p_{11}$ as p_{12}, $1 - p_{22}$ as p_{23}, $1 - p_{33}$ as p_{34}, $1 - p_{44}$ as p_{45}, and sets 1 to p_{55}. Now the rows are forced to add to 1.0.

9.2.2 Combination of Estimated Transition Probability Matrices for Different Time Steps

In reality, not all "before" and "after" inspections are done with an exactly same constant time difference or interval. For example, bridge inspections may be performed with several months apart to several years apart, although 2 years apart is the norm in the United States on average. Inspection data obtained with different time intervals should not be mixed in one estimation calculation as formulated in Equation 9.12. For example, three 1-year transition probability matrices multiplied with each other gives a 3-year matrix in concept, which should not be mixed with 1-year matrices.

Instead, the data need to be grouped according to the length of inspection interval. For each group with the same inspection interval, Equation 9.12 can be computed, which will result in **P** for that particular inspection interval or time step. To combine these transition probability matrices estimated using data with different time intervals, Pontis does offer a function to do just that, which is presented in the following discussion.

Based on the homogeneous Markov Chain concept, the transition probability matrix for n-step (n – time intervals) is defined as the product of n one-step (one-time interval) transition probability matrices:

$$\mathbf{P}^{n^T} = \underbrace{\mathbf{P}^T\ \mathbf{P}^T \ldots\ldots \mathbf{P}^T}_{n \text{ matrices multiplied}} \tag{9.13}$$

According to this concept, Pontis determines a transition probability matrix **P** for 1 year as one step by combining equivalent one-step (1-year) transition matrices. Each of the equivalent one-step matrices is obtained from an n-step (n-year) matrix. This weighted combination is done one row at a time because the weight for each row of each matrix can be different.

This process can be described as follows:

$$[\mathbf{P}]_{\text{row}_i} = [\mathbf{P}_\text{e}]_{\text{row}_i} w_{1_i} + [\mathbf{P}_\text{e}^2]_{\text{row}_i} w_{2_i} + [\mathbf{P}_\text{e}^3]_{\text{row}_i} w_{3_i} + \ldots\ldots + [\mathbf{P}_\text{e}^{10}]_{\text{row}_i} w_{10_i} \quad (i = 1, 2, 3, 4, 5) \tag{9.14}$$

where $\mathbf{P}_\text{e}$, $\mathbf{P}_\text{e}^2$, $\mathbf{P}_\text{e}^3$, …, and $\mathbf{P}_\text{e}^{10}$ are the transition probability matrices estimated using inspection data, respectively, with 1-year, 2-year, 3-year, …, and 10-year time intervals. In Equation 9.14, w_{1_i}, w_{2_i}, … w_{10_i} are weights for these 10 matrices and row I, respectively. They should satisfy

$$w_{1_i} + w_{2_i} + w_{3_i} + \ldots + w_{10_i} = 1 \tag{9.15}$$

Each of the transition probability matrices in Equation 9.14 for different time intervals can be expressed as follows with their transition probabilities identified:

$$\mathbf{P}_\text{e} = \begin{Bmatrix} p_{11} & 1-p_{11} & 0 & 0 & 0 \\ 0 & p_{22} & 1-p_{22} & 0 & 0 \\ 0 & 0 & p_{33} & 1-p_{33} & 0 \\ 0 & 0 & 0 & p_{44} & 1-p_{44} \\ 0 & 0 & 0 & 0 & p_{55} \end{Bmatrix} \tag{9.16}$$

$$\mathbf{P}_\text{e}^2 = \begin{Bmatrix} \sqrt{p_{11}^2} & 1-\sqrt{p_{11}^2} & 0 & 0 & 0 \\ 0 & \sqrt{p_{22}^2} & 1-\sqrt{p_{22}^2} & 0 & 0 \\ 0 & 0 & \sqrt{p_{33}^2} & 1-\sqrt{p_{33}^2} & 0 \\ 0 & 0 & 0 & \sqrt{p_{44}^2} & 1-\sqrt{p_{44}^2} \\ 0 & 0 & 0 & 0 & \sqrt{p_{55}^2} \end{Bmatrix} \tag{9.17}$$

$$\mathbf{P}_\text{e}^3 = \begin{Bmatrix} \sqrt[3]{p_{11}^3} & 1-\sqrt[3]{p_{11}^3} & 0 & 0 & 0 \\ 0 & \sqrt[3]{p_{22}^3} & 1-\sqrt[3]{p_{22}^3} & 0 & 0 \\ 0 & 0 & \sqrt[3]{p_{33}^3} & 1-\sqrt[3]{p_{33}^3} & 0 \\ 0 & 0 & 0 & \sqrt[3]{p_{44}^3} & 1-\sqrt[3]{p_{44}^3} \\ 0 & 0 & 0 & 0 & \sqrt[3]{p_{55}^3} \end{Bmatrix} \tag{9.18}$$

$$\vdots$$

$$\mathbf{P}_e^n = \begin{bmatrix} \sqrt[n]{p_{11}{}^n} & 1-\sqrt[n]{p_{11}{}^n} & 0 & 0 & 0 \\ 0 & \sqrt[n]{p_{22}{}^n} & 1-\sqrt[n]{p_{22}{}^n} & 0 & 0 \\ 0 & 0 & \sqrt[n]{p_{33}{}^n} & 1-\sqrt[n]{p_{33}{}^n} & 0 \\ 0 & 0 & 0 & \sqrt[n]{p_{44}{}^n} & 1-\sqrt[n]{p_{44}{}^n} \\ 0 & 0 & 0 & 0 & \sqrt[n]{p_{55}{}^n} \end{bmatrix} \tag{9.19}$$

where p_{11}, p_{22}, p_{33}, p_{44}, and p_{55} are diagonal terms of the transition probability matrix $\mathbf{P}_e$ estimated using inspection data spanning over 1 year. The regression procedure described earlier in Section 9.2.1 is used to find these probabilities. The probabilities p_{11}^2, $p^2{}_{22}$, $p^2{}_{33}$, $p^2{}_{44}$, and $p^2{}_{55}$ are obtained using data over 2 years for the "before" and "after" inspections. The same procedure in Section 9.2.1 is supposed to be used to find these terms. Please notice that the superscript "2" here indicates the time interval of 2 years, and it is not an exponent. Similarly p_{11}^n, $p^n{}_{22}$, $p^n{}_{33}$, $p^n{}_{44}$, and $p^n{}_{55}$ are the same except using data over n years.

It is seen in Equations 9.18 and 9.19 that the diagonal terms of the transition probability matrix for n years are taken nth root for $n = 2, 3, \ldots, 10$ to be combined with the 1-year matrix as defined in Equation 9.14. Furthermore, all other probabilities in the matrices are set to zero except the ones next to the diagonal terms to the immediate right, as discussed in Section 9.2.1. This is done based on two assumptions: (1) the condition will not improve without repair or rehabilitation and (2) deterioration will not take place in the form of skipping a condition state (i.e., from state 1 to 3, from state 2 to 4, or from state 3 to 5). The second assumption may be true for short time periods such as 1 or 2 years, but questionable for longer periods such as 8, 9, and 10 years. Practically, however, this is not a serious concern at this point, because perhaps no bridge was inspected that many years apart. Nevertheless, transition probabilities over 3 or 4 years may very well be nonzero skipping a state. Application examples show that ignoring these (i.e., setting them to zero) apparently will result in a lower deterioration rate (Fu and Devaraj 2008).

The weights in Equation 9.14 are set in Pontis as follows, depending on the number of data pairs used to estimate $\mathbf{P}_e$, $\mathbf{P}_e{}^2$, $\mathbf{P}_e{}^3$, …, $\mathbf{P}_e{}^{10}$ for each row, respectively

$$w_{ji} = \frac{N_{ji}}{\sum_k N_{ki}} \quad i = 1, 2, 3, \ldots, 5; \quad j = 1, 2, 3, \ldots, 10 \tag{9.20}$$

where N_{ji} is the number of data pairs with j-years apart and transition (deterioration) starting from state i, used to estimate probability matrix $\mathbf{P}_e{}^i$. In other words, the more data pairs are used for a transition probability matrix, the heavier the resulting matrix will be weighted when combined with other matrices for different years. Also, the more data pairs there are for a row in a matrix, that row will be weighted more when combined with the same row in other matrices.

Note that the combined resulting transition matrix is to be applied to the same element from all the bridges in the same environment and for all the future years, based on the homogenous Markov Chain assumption. Since that matrix is for one-step equal to 1 year, for n-step (n-year) transition, n 1-year matrices will be multiplied to obtain the n-year transition probability matrix. Then, it is multiplied by the corresponding initial distribution to find the predicted condition distributions in the future.

9.3 Nonhomogeneous Markov Chain Model

In a nonhomogeneous Mark Chain model, the transition probability matrix is not constant but a function of time. Age is viewed here as a relative measure of time, independent of the absolute time. For bridge management application, we propose to take into account the bridge element age. An optimization approach is proposed below to estimate the nonhomogeneous transition probabilities. Namely, the error or the difference between the predicted and observed and measured bridge element conditions is to be minimized. The prediction process no longer assumes a constant transition probability matrix as for the homogeneous Markov Chain. Instead, the following formulation is developed for estimating or identifying the age-dependent (i.e., nonhomogeneous) transition probability matrices for each bridge element:

$$\text{Minimize } \Sigma_{i=1,\ldots,N}\left|\boldsymbol{Y}_i - \text{Predicted}\left[\boldsymbol{Y}_i, \boldsymbol{P}\left(A_i\right)\right]\right|^2 \tag{9.21}$$

$$\text{Subject to } \Sigma_{k=1,2,\ldots,S}\, p_{jk}(A_i = 1 \text{ for all } j) \tag{9.22}$$

$$0 \leq p_{jk}(A_i) \leq 1 \text{ for all } k \text{ and } j \tag{9.23}$$

where N is the total number of condition transition data pairs used; $\boldsymbol{Y}_i$ is the condition state vector right after the ith transition for the corresponding data pair; Predicted $[\boldsymbol{Y}_i, \boldsymbol{P}(A_i)]$ is the predicted condition state vector for the same element involved in the ith transition, using the transition probability matrix $\boldsymbol{P}(A_i)$ depending on the element's age A_i. The symbol $\left|\boldsymbol{Y}_i - \text{Predicted}\left[\boldsymbol{Y}_i, \boldsymbol{P}(A_i)\right]\right|$ means the magnitude or modulus of vector $\boldsymbol{Y}_i - \text{Predicted}\,[\boldsymbol{Y}_i, \boldsymbol{P}(A_i)]$. The transition probabilities p_{jk} for j, k = 1, 2, …, S are the elements of matrix $\boldsymbol{P}$, and S is the total number of possible states for that element. The constraints for them to satisfy in Equations 9.22 and 9.23 are for consistency for the same function of Equation 9.21. These conditions are enforced in the process of estimating p_{jk}, treated differently from the Pontis approach as commented in Section 9.2.1.

The Markov Chain model used here is more general than the Pontis' homogeneous model, for its nonhomogeneity. Therefore, the transition probability matrices $\boldsymbol{P}(A)$ are shown as functions of age A. It means that $\boldsymbol{P}(A_i)$ can be different according to age A_i of the element involved in transition i. For application to bridge management focused herein, we take into account the effect of element age. This can be seen more easily in application examples given in Sections 9.3.1 and 9.3.2.

Practically, bridge inspection data are not strictly evenly intervaled. Equations 9.21 through 9.23 can readily accommodate such data, such as those intervaled by 2, 3, or any other number of years. In Pontis, in contrast, data from unevenly intervaled inspections are grouped according to the inspection interval, because mixing them may cause excessive approximation. Each group is used separately for estimating the transition probability matrix for that group. Then the resulting matrices for different groups are averaged with weights proportional to the number of data pairs used in each group.

9.3.1 Application Example Element 215 (Reinforced Concrete Abutment)

The condition inspection data of the bridge element are taken from the Pontis database of Michigan Department of Transportation (MDOT). This element is defined to have four states, instead of five. Before using the data, we first examined the data cleaning procedure used in Pontis and found that the process is not consistent enough because it misses invalid pairs. A typical example pair that has passed the Pontis screening is given in Table 9.1.

TABLE 9.1 Condition State Distribution for Element 215 of MDOT Bridge *27127022000B030* in Environment 1

$X^{(0)}$	$x_1^{(0)}$	$x_2^{(0)}$	$x_3^{(0)}$	$x_4^{(0)}$
Percentage	77.778	14.815	7.407	0
$Y^{(1)}$	$y_1^{(1)}$	$y_2^{(1)}$	$y_3^{(1)}$	$y_4^{(1)}$
Percentage	0	96.296	3.704	0

In this table, vectors $\boldsymbol{X}^{(0)} = (x_1^{(0)}, x_2^{(0)}, \ldots, x_S^{(0)})$ and $\boldsymbol{Y}^{(1)} = (y_1^{(1)}, y_2^{(1)}, \ldots, y_S^{(1)})$ are the condition state distribution vectors from the previous and current inspection, respectively. Note that no improvement work was done between the two times when $\boldsymbol{X}^{(0)}$ and $\boldsymbol{Y}^{(1)}$ were obtained through inspection. Thus, the sum of $x_1^{(0)}$ and $x_2^{(0)}$ for condition states 1 and 2 should be greater than or equal to the sum of $y_1^{(1)}$ and $y_2^{(1)}$ because the latter two can only come from the former two, based on an assumption used in Pontis that no skipping of condition state is accepted between two inspections. However, this is not true as seen in Table 9.1, as the sum of $x_1^{(0)}$ and $x_2^{(0)}$ is 92.593, which is less than the sum of $y_1^{(1)}$ and $y_2^{(1)}$, which is 96.296. These inconsistent data may be due to an input error such as typo.

To improve data cleaning, a more rigorous procedure was developed in this research effort (Fu and Devaraj 2008), which included checks for all logical relations between the values as noted in the preceding discussion. This new procedure has been used in the examples presented here to screen out invalid data pairs that were not identified by the Pontis screening process.

For estimating the transition probability matrices, Equations 9.21 through 9.23 are applied to this example using all available MDOT data for this element in one environment in Michigan up to year 2000. Three transition probability matrices $\boldsymbol{P}$ are used here to model the element's deterioration, denoted as $\boldsymbol{U}_{0-20}$, $\boldsymbol{V}_{21-40}$, and $\boldsymbol{W}_{41-}$, respectively covering age ranges of 0–20 years, 21–40 years, and 41 years and beyond. Note that Equations 9.21 through 9.23 are flexible for any age group deemed to be reasonable. Accordingly, Equations 9.21 through 9.23 are specifically written as follows for this example.

$$\begin{aligned}\text{Minimize}\ \{&\Sigma_{i=1,\ldots,N;0-20}\left|\boldsymbol{Y}_i^{(1)} - \boldsymbol{X}_i^{(0)}\boldsymbol{U}_{0-20}\right|^2\\ &+\Sigma_{i=1,\ldots,N;21-40}\left|\boldsymbol{Y}_i^{(1)} - \boldsymbol{X}_i^{(0)}\boldsymbol{V}_{21-40}\right|^2\\ &+\Sigma_{i=1,\ldots,N;41-}\left|\boldsymbol{Y}_i^{(1)} - \boldsymbol{X}_i^{(0)}\boldsymbol{W}_{41-}\right|^2\}\end{aligned} \tag{9.24}$$

$$\begin{aligned}\text{Subject to}\ &\Sigma_{k=1,2,\ldots,4}\, u_{jk,\,0-20} = 1 \text{ for all } j\\ &\Sigma_{k=1,2,\ldots,4}\, v_{jk,\,21-40} = 1 \text{ for all } j\\ &\Sigma_{k=1,2,\ldots,4}\, w_{jk,\,41-} = 1 \text{ for all } j\end{aligned} \tag{9.25}$$

$$\begin{aligned}&0 \le u_{jk,0-20} \le 1 \text{ for all } k \text{ and } j\\ &0 \le v_{jk,21-40} \le 1 \text{ for all } k \text{ and } j\\ &0 \le w_{jk,41-} \le 1 \text{ for all } k \text{ and } j\end{aligned} \tag{9.26}$$

where bolded capital letters denote vectors or matrices and lowercase letters not bolded are scalars as the elements in the transition probability matrices. For example, $u_{jk,\ 0-20}$, $v_{jk,\ 21-40}$, and $w_{jk,\ 41-}$ are probabilities in matrices $\boldsymbol{U}_{0-20}$, $\boldsymbol{V}_{21-40}$, and $\boldsymbol{W}_{41-}$ for transition from state j to k, respectively. Also, N_{0-20}, N_{21-40}, and N_{41-} are, respectively, the numbers of data pairs within each age group indicated by the subscript. In addition, the following conditions are used as in Pontis. (1) For the do-nothing situation, the probabilities for any condition improvement are zero. (2) Over an inspection interval, condition transition can only occur between two consecutive states. In other words, no skipping of state is accepted, which is also included in the data cleaning procedure (Fu and Devaraj 2008). (3) The worst state 4 is an absorbing state with the probability p_{jj} (j = 1, 2, 3, 4) equal to 1. These conditions are explicitly given in the following for a complete presentation.

$$u_{jk,0-20} = 0 \text{ if } j > k; \quad u_{jk,0-20} = 1 - u_{jj,0-20} \quad \text{if } k - j = 1 \\ \qquad\qquad = 0 \qquad \text{otherwise} \tag{9.27}$$

$$v_{jk,21-40} = 0 \text{ if } j > k; \quad v_{jk,21-40} = 1 - v_{jj,21-40} \quad \text{if } k - j = 1 \\ \qquad\qquad = 0 \qquad \text{otherwise} \tag{9.28}$$

$$w_{jk,41-} = 0 \text{ if } j > k; \quad w_{jk,41-} = 1 - w_{jj,44-} \quad \text{if } k - j = 1; \\ \qquad\qquad = 0 \qquad \text{otherwise} \tag{9.29}$$

$$u_{44,0-20} = v_{44,21-40} = w_{44,41-} = 1 \tag{9.30}$$

This makes only the first three diagonal terms in each of the three transition probability matrices ($\boldsymbol{U}_{0-20}$, $\boldsymbol{V}_{21-40}$, and $\boldsymbol{W}_{41-}$) unknown to be found in the minimization process: $u_{11,\ 0-20}$; $u_{22,\ 0-20}$; $u_{33,\ 0-20}$; $v_{11,\ 21-40}$; $v_{22,\ 21-40}$; $v_{33,\ 21-40}$; $w_{11,\ 41-}$; $w_{22,\ 41-}$; and $w_{33,\ 41-}$. Using inspection data up to year 2000, Table 9.2 displays these 9 terms for environment 1 in Michigan for this application example, along with the last terms in the probability matrices ($u_{44} = v_{44} = w_{44} = 1$). Since this last term is not treated as an unknown for the nonhomogeneous Markov Chain approach, it is given in bold. These nine terms represent the critical part of the transition probability matrices $\boldsymbol{U}_{0-20}$, $\boldsymbol{V}_{21-40}$, and $\boldsymbol{W}_{41-}$. They are compared in Table 9.2 with the constant transition probability matrix obtained using the Pontis approach.

It is seen in Table 9.2 that the Pontis transition probabilities are mostly between the maximum and minimum values of those in the matrices $\boldsymbol{U}_{0-20}$, $\boldsymbol{V}_{21-40}$, and $\boldsymbol{W}_{41-}$. This actually highlights the essence of the homogeneous Markov Chain application here—modeling a nonhomogeneous Markov Chain with compromise. Constrained by the homogeneity assumption, it would not be able to realistically model the nonhomogeneous stochastic process. For projecting to a remote future (tens or hundreds of years) for bridge management, the discrepancy between reality and the homogeneity assumption can become significant and unacceptable.

For other three environments, similar comparisons are observed, showing the Pontis-obtained transition matrices to be a compromise to fit the data of real observations. This is seen more clearly in Table 9.3 when the nonhomogeneous Markov Chain approach used six transition probability matrices

TABLE 9.2 Comparison of Transition Probability Matrices between Proposed Method Using Three Matrices and Pontis Approach for Element 215 in Environment 1

	p11	p22	p33	**p44**
$\boldsymbol{U}_{0-20}$	0.977	0.986	1.000	**1.000**
$\boldsymbol{V}_{21-40}$	0.968	0.986	0.941	**1.000**
$\boldsymbol{W}_{41-}$	0.960	0.982	1.000	**1.000**
Pontis	0.963	0.985	0.990	**1.000**

TABLE 9.3 Comparison of Transition Probability Matrices between Proposed Method Using Six Matrices and Pontis Approach for Element 215 in Environment 1

	p11	p22	p33	**p44**
$\boldsymbol{U}_{0-10}$	0.972	0.989	0.995	**1.000**
$\boldsymbol{V}_{11-20}$	0.989	0.980	0.990	**1.000**
$\boldsymbol{W}_{21-30}$	0.973	0.959	0.990	**1.000**
$\boldsymbol{X}_{31-40}$	0.967	0.992	0.978	**1.000**
$\boldsymbol{Y}_{41-50}$	0.945	1.000	0.990	**1.000**
$\boldsymbol{Z}_{51-}$	0.967	0.976	0.999	**1.000**
Pontis	0.963	0.985	0.990	**1.000**

$\boldsymbol{U}$, $\boldsymbol{V}$, $\boldsymbol{W}$, $\boldsymbol{X}$, $\boldsymbol{Y}$, and $\boldsymbol{Z}$ to, respectively, cover age ranges of 0–10, 11–20, 21–30, 31–40, 41–50, and beyond 50 years. This choice of more nonhomogeneous transition probability matrices further improves modeling.

Evaluation of the proposed nonhomogeneous Markov Chain approach is also conducted in this study using the following procedure. First, the estimated transition probabilities are used to predict the element's immediate future distribution vector at the network level. Then this predicted distribution is compared with the measured distribution vector using the inspection data. A relative error is then calculated to quantitatively evaluate the effectiveness of prediction. A smaller error indicates better prediction that is desired. It is important to emphasize that this future distribution vector based on inspection results was not used in the process of estimating the transition probabilities. Essentially, this evaluation simulates a practical application of using the latest inspection data to predict the future bridge condition at the network level, which is often practiced among agencies.

Table 9.4 displays the results of the evaluation for this example using three matrices ($\boldsymbol{U}_{0-20}$, $\boldsymbol{V}_{21-40}$, and $\boldsymbol{W}_{41}$) compared with those for the Pontis approach, using Element 215 data of MDOT beyond year 2000. All four environments in Michigan are included. Two errors are shown in Table 9.4, one using data up to year 2002 for evaluation and the other up to year 2004. Largely as expected, the proposed approach has produced smaller errors, mainly due to the higher modeling resolution using several transition probability matrices for the entire life span of the element. This is also seen more clearly in Table 9.5 for the same comparison but using six matrices, where nonhomogeneity is modeled with higher resolution. It is seen that using six transition probability matrices (each covering 10 years of age except the last matrix) has performed generally better than using three matrices (each covering 20 years of age except the last matrix).

Note also that when year 2002 data were used for evaluation, inspection data up to year 2000 were used for estimating the transition probability matrices. When year 2004 data were used for evaluation, inspection data up to year 2002 were used for estimation. Again this is to simulate a case of realistic practice of bridge management.

It is worth mentioning that, again, more transition probability matrices can be used in the proposed nonhomogeneous Markov Chain model to improve modeling when warranted. Of course, this will increase the requirement for computation effort as seen in Equations 9.21 through 9.23.

As mentioned in Section 9.2.1, the Pontis approach for estimating the transition probability matrix may cause probability values to become negative, above unity, and/or the sum of a row not to add to 1, especially when the number of valid data pairs is small. This problem is now completely resolved in the optimization problem formulated in Equations 9.21 through 9.23, as well as Equations 9.24 through 9.30 for this particular example. As discussed in Section 9.2.1, the Pontis approach represents a different treatment of the same subjects and may cause significant approximation and error. Therefore, that approach is not expected to produce consistently reliable results every time when applied. This is seen in the results in Tables 9.4 and 9.5.

TABLE 9.4 Comparison of Errors (%) for Proposed Method Using Three Matrices and Pontis Approach for Element 215

		Using 2002 Data for Evaluation	*Using 2004 Data for Evaluation*
Env1	Proposed approach	2.11	3.05
	Pontis approach	2.89	3.88
Env2	Proposed approach	3.12	3.95
	Pontis approach	3.83	4.48
Env3	Proposed approach	5.67	1.01
	Pontis approach	7.11	2.03
Env4	Proposed approach	1.95	6.10
	Pontis approach	2.82	6.97

TABLE 9.5 Comparison of Errors (%) for Proposed Method Using Six Matrices and Pontis Approach for Element 215

		Using 2002 Data for Evaluation	*Using 2004 Data for Evaluation*
Env1	Proposed approach	2.04	3.08
	Pontis approach	2.89	3.88
Env2	Proposed approach	2.89	2.99
	Pontis approach	3.83	4.48
Env3	Proposed approach	6.81	0.94
	Pontis approach	7.11	2.03
Env4	Proposed approach	1.50	5.81
	Pontis approach	2.82	6.97

TABLE 9.6 Comparison of Errors (%) for Proposed Method Using Three Matrices and Pontis Approach for Element 104

		Using 2002 Data for Evaluation	*Using 2004 Data for Evaluation*
Env1	Proposed approach	0.06	0.01
	Pontis approach	0.06	0.35
Env2	Proposed approach	0.26	0.07
	Pontis approach	0.78	0.03
Env3	Proposed approach	0.69	0.35
	Pontis approach	1.85	1.04
Env4	Proposed approach	0.48	0.34
	Pontis approach	0.44	0.36

9.3.2 Application Example Element 104 (Prestressed Concrete Box Beam)

Tables 9.6 and 9.7 show the evaluation results for this element in Michigan, using data from the Pontis database of MDOT.

It is shown in Table 9.6 that the first set of results for environment 1 using year 2002 data for evaluation involves the same error for both approaches used. The reason for this is that there is no deterioration for all the data pairs used in estimating the transition probabilities. To be exact, there are 38 data pairs for this environment, and all of them have 100% of the element in state 1 before and after inspection. Thus, the estimations of the transition probabilities or their interpretations using the two

TABLE 9.7 Comparison of Errors (%) for Proposed Method Using Six Matrices and Pontis Approach for Element 104

		Using 2002 Data for Evaluation	*Using 2004 Data for Evaluation*
Env1	Proposed approach	0.06	0.02
	Pontis approach	0.06	0.35
Env2	Proposed approach	0.27	0.10
	Pontis approach	0.78	0.03
Env3	Proposed approach	0.27	0.01
	Pontis approach	1.85	1.04
Env4	Proposed approach	0.46	0.17
	Pontis approach	0.44	0.36

different methods lead to the same result. In other words, the data show no deterioration, and these different methods have consistent interpretation for the underlying nondeteriorating mechanism. The 0.06% error in Table 9.6 is simply due to the inconsistency of the future (year 2002) data with those used in estimating the transition probabilities.

Table 9.7 displays the same comparison but using six different transition matrices in the proposed method based on a nonhomogeneous Markov Chain. In general, these results show improvement from those in Table 9.6 with reduced errors for the proposed method.

9.4 Summary

A large majority of state transportation agencies use Pontis, although the level of experience varies. The most experienced states have collected more than 10 years of condition inspection data. The level of satisfaction with Pontis also varies among the states. A critical issue is the estimation of the transition probability matrices, which model deterioration of bridge elements. The Pontis BMS has the following issues to be addressed: (1) possible negative and larger than unity transition probabilities from the regression process, although respectively set to zero and unity when found; (2) possible transition probabilities in one row of the matrix that do not add to 1, although enforced when found so; (3) inadequate consistency screening of raw data for more reliable estimation results; and (4) assumed homogeneity of Markov Chain.

A nonhomogeneous Markov Chain model has been proposed in this chapter, for improving modeling bridge element deterioration and improvement. The homogeneous Markov Chain model used in Pontis can be viewed as a special case of this new model and approach. Application examples show that it can better predict bridge element deterioration trends.

Acknowledgments

The work reported here was funded by the Michigan Department of Transportation and Federal Highway Administration. This support is gratefully appreciated.

References

AASHTO. 2005. *Pontis Release 4.4, Technical Manual*, American Association of State Highway and Transportation Officials, DC.

Baik, H., Jeong, H.S., and Abraham, D.M. 2006. "Estimating Transition Probabilities in Markov Chain-Based Deterioration Models for Management of Wastewater Systems," *J. Water Resour. Plann. Manage.*, 132(1), 15–24.

Fu, G. 1987. "Modeling of Lifetime Structural System Reliability," Ph.D. Dissertation, Report No.87-9, Department of Civil Engineering, Case Western Reserve University, Cleveland, OH.

Fu, G., and Devaraj, D. 2008. "Methodology of Homogeneous and Non-homogeneous Markov Chains for Modeling Bridge Element Deterioration," *Final Report to Michigan Department of Transportation, MDOT Research Report RC-1520*, Center for Advanced Bridge Engineering, Department of Civil and Environmental Engineering, Wayne State University, August, Detroit, MI.

Fu, G., and Moses, F. 1986. "Application of Lifetime System Reliability," *Preprint No.52-1*, American Society of Civil Engineers—Structures Congress'86, New Orleans, LA, September 15–18, p. 281.

Puterman, M.L. 1994. *Markov Decision Processes*, John Wiley & Sons, Inc, New York, NY.

10

Bridge Health Monitoring

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10.1 Introduction

Over the last two decades, structural health monitoring (SHM) technologies have emerged, creating an attractive field within bridge engineering. Application of bridge health monitoring (BHM) has been recognized as an attractive tool for early warnings on structural damage or deterioration before costly maintenance and repair, or even unexpected bridge collapse (Brownjohn 2007). To successfully accomplish the purposes of BHM, several challenging tasks from the selection of proper sensors to the development of effective algorithms for monitored data analysis and interpretation were investigated. The need for a bridge health paradigm that helps to effectively manage bridge performance was highlighted (Ko and Ni 2005; Catbas et al. 2008, 2013a, 2013b; Frangopol 2011). With a sufficient amount of monitored data, bridge performance under uncertainty can be predicted reliably and, furthermore, bridge maintenance management can be optimized to allocate limited financial resources and extend the service life of bridges (Strauss et al. 2008; Messervey et al. 2011; Orcesi and Frangopol 2013). Therefore, probabilistic and statistical concepts and methods, time-dependent bridge performance under uncertainty, and bridge life-cycle cost analysis and performance prediction should be well understood to achieve the recent need for a bridge health paradigm.

In this chapter, the general role of SHM for bridge performance management under uncertainty is emphasized. The effect of BHM on bridge performance management under uncertainty is presented. For efficient use of monitoring data and cost-effective monitoring planning, probabilistic approaches based on reliability, statistics of extremes, availability, and damage detection delay are provided. Finally, the integrated life-cycle framework using bridge monitoring for improved bridge management under uncertainty is discussed. The concepts and approaches presented in this chapter can serve as a fundamental background for further advances in bridge management under uncertainty using BHM information.

10.2 Bridge Health Monitoring and Performance Management

The purpose of BHM is to quantify bridge responses to external loadings. To obtain reliable monitoring data, the monitoring system has to be installed effectively and efficiently, and the monitoring duration has to be sufficient. Significant developments in SHM technologies have been achieved and applied to a wide range of civil infrastructure systems (Brownjohn 2007). Despite the progress attained in developing advanced SHM technologies, statistical and probabilistic approaches for SHM data acquisition and interpretation and an integrated life-cycle framework for optimal bridge monitoring and maintenance under uncertainty have to be developed and implemented (Frangopol 2011).

10.2.1 General Objectives of Bridge Health Monitoring

The general objectives of BHM include (1) detecting and identifying the damages at potential critical locations of a bridge component, (2) assessing and predicting the structural performance, (3) providing the database for optimum bridge maintenance intervention, and (4) improving the bridge design process (Farrar and Lieven 2007; Farrar and Worden 2007; Frangopol et al. 2008b; Liu et al. 2009b).

BHM is applicable to bridges under construction or in service. For example, the Lehigh River Bridge SR-33, located in Bethlehem, Pennsylvania, was monitored by the Advanced Technology for Large Structural Systems (ATLSS) Engineering Research Center during several construction phases (from August 2001 to October 2001) and after the construction period (from January 2002 to March 2005). The main objectives of this monitoring program were (1) investigating the long-term effects of creep and shrinkage of the concrete deck and temperature changes on the truss, (2) understanding better the composite truss–concrete deck interaction during the construction period, and (3) investigating long-term behavior under various loading effects while in service. Detailed descriptions and results of this monitoring program can be found in the works of Connor and Santosuosso (2002) and Connor and McCarthy (2006).

Degradation of bridge performance may be caused by the combined effects of structural aging, aggressive environmental stressors, and loading conditions under uncertainty. BHM has a great potential in performance assessment and prediction, and cost-effective maintenance by reduction of uncertainty. This reduction can lead to preventing the unexpected failure of a structure, assessing and predicting structural performance more reliably, and applying appropriate maintenance on time (Frangopol 2011).

10.2.2 Bridge Health Monitoring Application Example

10.2.2.1 Bridge Description

As an illustrative example, the BHM for Birmingham Bridge over the Monongahela River is presented. This bridge, located in Pittsburgh, Pennsylvania, was built in 1976. It is a tied arch bridge having a main span length of 189 m (620 ft.). According to Connor et al. (2004), fatigue cracks have been observed in almost all of the transverse floor beams at the connection to tie girders over the several past years. The ATLSS Engineering Research Center at Lehigh University inspected this connection to identify the cause of fatigue cracking, and, in 2002, proposed appropriate retrofit options (Connor and Fisher 2002). Finally, it was concluded that the cracking was caused by relative displacement occurring between the floor-beam web connected to the tie girder and the flange (see Figure 10.1a). Although this displacement is very small, it can lead to a high stress within the web gap. Softening the connection by removing a portion of the floor-beam flange and web near the tie girder was done as a retrofit in order to prevent the occurrence of high stress (see Figure 10.1b); no further crack development occurred. The cutout region with the instrumentation plan was monitored from October to December, 2003.

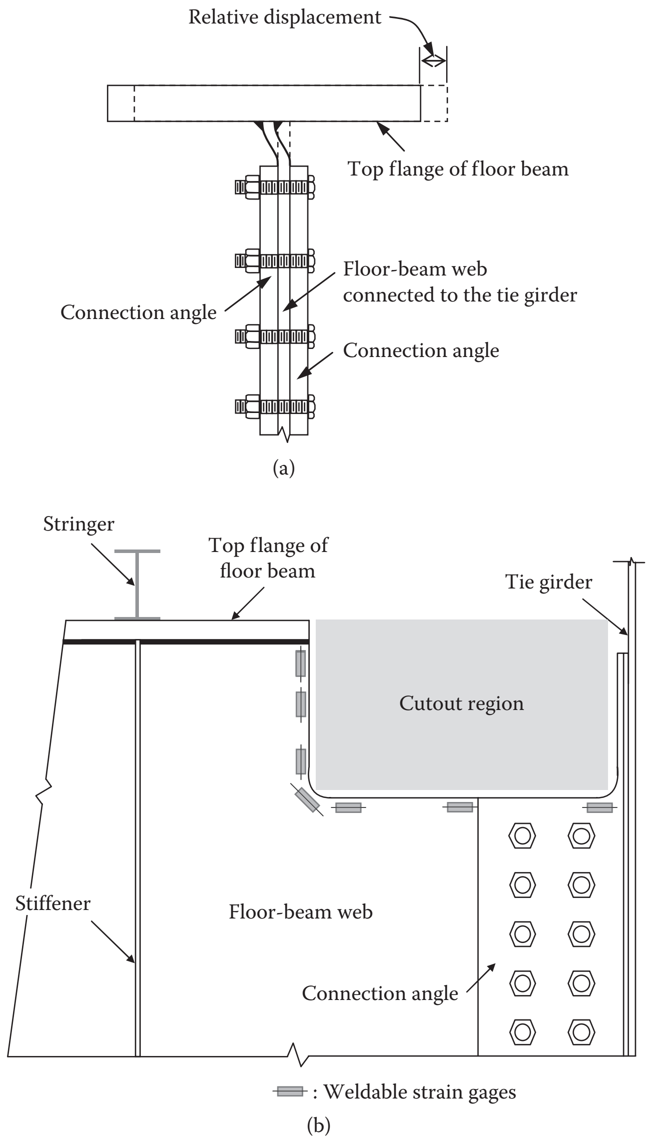

FIGURE 10.1 Schematic showing (a) relative displacement occurring between the floor beam and the tie girder and (b) floor-beam connection details after retrofitting and placing the sensors. (Adapted from Connor, R.J. et al., *ATLSS Report No. 04-04*, Lehigh University, Bethlehem, Pennsylvania, 2004.)

10.2.2.2 Sensor Selection and Installation

The objective of BHM includes verifying the adequacy of the applied retrofit and evaluating the fatigue performance of the retrofitted connection detail between the floor beam and the tie girder (Connor et al. 2004). In this BHM, a total of 32 weldable strain gages were installed on the ends of the floor beam adjacent to the connection to the tie girder to measure the relative displacement, as well as the bottom flange of a floor beam and the tied girder at midspan to calibrate the global displacement. Figure 10.1b shows the sensor locations on the floor beam adjacent to the connection to the tie girder. Considering the various weather conditions, weldable gages, which are temperature-compensated uniaxial strain gages, were used in this BHM. Detailed information on the sensor installation can be found in the work by Connor et al. (2004).

10.2.2.3 Data Acquisition and Management

BHM data were continuously collected during a monitoring period of almost 40 days. A high-speed, multichannel, 16-bit digital data acquisition system was employed, and in this system analog filtering and digital filtering were applied to collect stable and noise-free data. A high-speed wireless Internet connection was used to manage the monitored data by the ATLSS Engineering Research Center at Lehigh University in real time remotely (Connor et al. 2004). The collected BHM data were saved as stress versus time (i.e., time history stress) and transferred into stress range versus number of cycles by using the rainflow counting method. In order to consider only the heavy vehicles, stresses were recorded only when the stress induced by the vehicle exceeded the predefined stress. The transferred data were used to estimate the fatigue performance of the bridge. Several approaches for fatigue reliability assessment and prediction using these BHM data were introduced by Liu et al. (2010) and Kwon and Frangopol (2010).

10.2.3 Bridge Performance Assessment and Prediction under Uncertainty

Understanding deterioration mechanisms and predicting the accurate performance of a deteriorating bridge are essential for ensuring an acceptable level of structural safety and establishing cost-effective maintenance strategies. The causes of bridge performance degradation are mainly resistance reduction caused by aging and aggressive environmental stressors, and increase in magnitude and frequency of external loadings over time (Enright and Frangopol 1998a; Frangopol and Kim 2011).

The resistance reduction of concrete and steel bridges is generally related to corrosion and fatigue. Corrosion and fatigue are highly dependent on various environmental conditions under uncertainty. External loadings are due to random truck weights, temperature, wind, and earthquake, among others. For this reason, it is difficult to accurately assess and predict the structural performance of a deteriorating bridge. Several studies focusing on probabilistic service life prediction have been conducted on deteriorating concrete bridges subject to corrosion (Frangopol et al. 1997; Enright and Frangopol 1998b; Stewart 2004; Li et al. 2005). Considering the uncertainties associated with loading conditions, environmental stressors, fabrication, and modeling of steel structures subjected to fatigue, several probabilistic approaches for assessing and predicting fatigue performance have been developed (Lukić and Cremona 2001; Cheung and Li 2003; Righiniotis and Chryssanthopoulos 2003). Time-dependent structural performance index under uncertainty is illustrated in Figure 10.2. By considering the uncertainties in modeling deterioration mechanisms and their associated parameters, the probability density functions (PDFs) of initial bridge performance index, $\mathrm{PI}_{\mathrm{ini}}$, and performance index at time t, PI_t, can be obtained as shown in this figure. The PDFs of damage initiation time, t_{ini}, and service life, t_{life} (i.e., the time when the bridge performance index reaches a predefined threshold, $\mathrm{PI}_{\mathrm{thres}}$), are also provided in Figure 10.2.

10.2.4 Effect of Monitoring on Bridge Performance Assessment and Prediction

The accuracy of bridge performance assessment and prediction can be improved by using the monitored data appropriately and efficiently. Possible effects on bridge performance assessment and prediction due to updating the damage initiation time, t_{ini}, using monitoring data are qualitatively shown in Figure 10.3. For illustrative purposes, three cases (i.e., case 1, case 2, and case 3) are considered in this figure assuming that the degradation rate of bridge performance, r_{p}, after damage initiation is constant and identical for all these cases. Case 1 represents the initial bridge performance index without updating the damage initiation time. Cases 2 and 3 are associated with updating based on monitoring data. Case 2 shows that if the damage initiation time is updated using monitoring data, and the mean of updated damage initiation time, $E(t_{\mathrm{ini},2})$, is less than the mean of initially predicted damage initiation time, $E(t_{\mathrm{ini},1})$, the mean of service life, $E(t_{\mathrm{life},2})$, will be less than the initially predicted service life, $E(t_{\mathrm{life},1})$; in this case, updating leads to timely maintenance action, preventing unacceptable performance. Case 3 indicates that updating the damage initiation time based on monitoring leads to larger mean values of

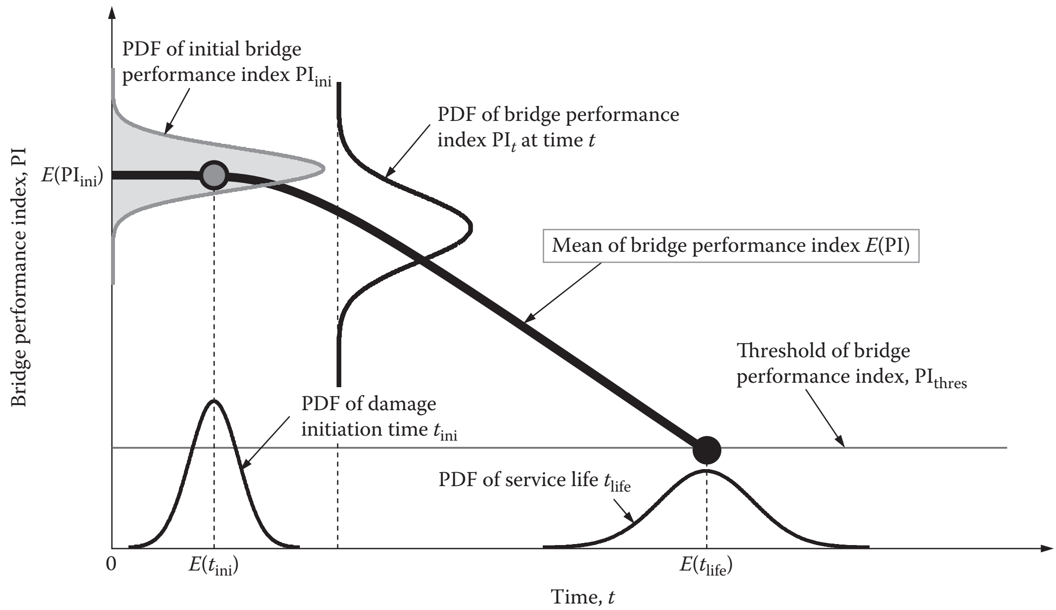

FIGURE 10.2 Time-dependent bridge performance index under uncertainty.

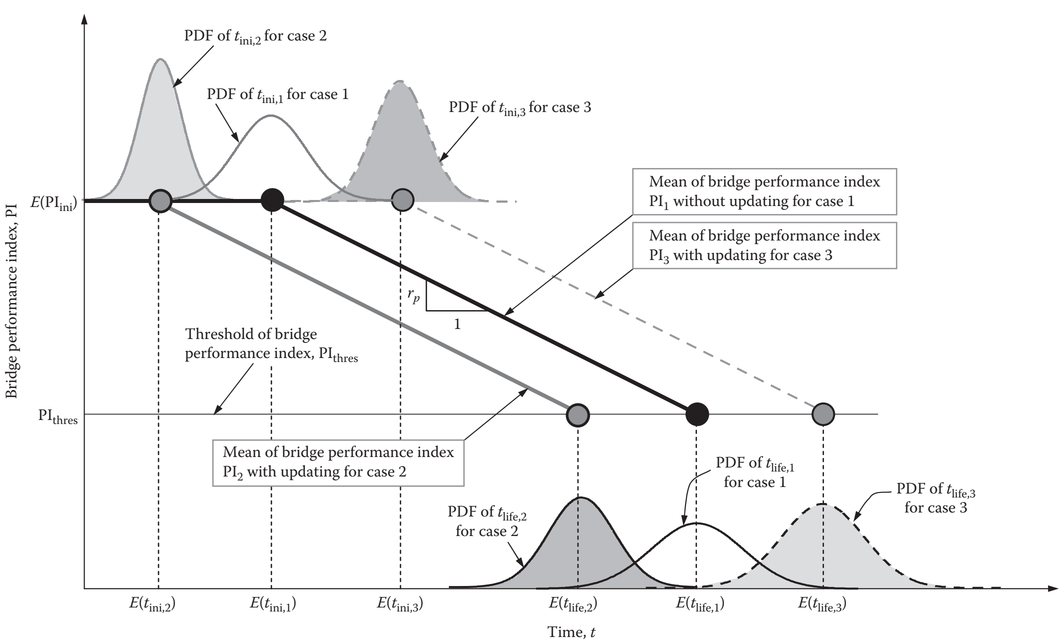

FIGURE 10.3 Effect of updating damage initiation time using monitoring data on bridge performance index under uncertainty.

damage initiation time $E(t_{ini,3})$ and service life $E(t_{life,3})$ than $E(t_{ini,1})$ and $E(t_{life,1})$, respectively; in this case, there will be financial benefit by avoiding unnecessary maintenance actions (Frangopol 2011). From this figure, it can be seen that even if the bridge is monitored before damage initiation, information from monitoring can be used to predict more accurate structural performance, resulting in preventing unacceptable performance or avoiding unnecessary maintenance actions. Figure 10.3 is based on only updating a single parameter (i.e., damage initiation time t_{ini}) related to bridge performance prediction. Updating multiple parameters (i.e., t_{ini} and r_p) will result in a more rational decision than that associated with updating a single parameter (Zhu and Frangopol 2013). Additional information concerning the effect of bridge monitoring and updating on bridge performance assessment and prediction can be found in Messervey (2008).

10.3 Efficient Use of Monitoring Data for Cost-Effective Bridge Management

Bridge life-cycle management can benefit from SHM through the efficient use of monitoring data (Frangopol 2011). Therefore, it is crucial to study a methodology to process the monitored data efficiently.

10.3.1 Effect of SHM on Bridge Life-Cycle Cost

Determining the types and times of maintenance actions is based on single- or multiobjective optimization associated with minimization of maintenance cost, maximization of structural performance during the service life of a bridge, and maximization of bridge service life. Minimizing the expected total life-cycle cost has generally been treated as the optimization criterion for life-cycle analysis (Frangopol et al. 1997; Frangopol and Maute 2003; Hassanain and Loov 2003; Kong and Frangopol 2005). The expected total life-cycle cost C_{total} is obtained as follows (Frangopol et al. 1997):

$$C_{\text{total}} = C_{\text{ini}} + C_{\text{main}} + C_{\text{insp}} + C_{\text{fail}} \tag{10.1}$$

where C_{ini} is the initial design and construction cost, C_{main} is the expected cost of maintenance, C_{insp} is the expected cost of inspection, and C_{fail} is the expected cost of failure. The expected life-cycle cost considering SHM can be expressed as follows (Frangopol and Messervey 2011):

$$C^*_{\text{total}} = C^*_{\text{ini}} + C^*_{\text{main}} + C^*_{\text{insp}} + C^*_{\text{fail}} + C_{\text{mon}} \tag{10.2}$$

where C_{mon} is the monitoring cost. The superscript "*" in Equation 10.2 denotes the cost in Equation 10.1 affected by SHM. The monitoring cost consists of cost of design and installation of SHM, operation cost, and management and maintenance cost. The benefit of SHM C_{ben} can be determined by subtracting Equation 10.2 from Equation 10.1 (i.e., $C_{\text{ben}} = C_{\text{total}} - C^*{}_{\text{total}}$). Figure 10.4 illustrates the relation between total life-cycle cost and lifetime bridge performance index with and without considering monitoring. A comparison between Figure 10.4a and b indicates that if monitoring is applied efficiently for bridge maintenance management the minimum total life-cycle cost with cost-effective monitoring, $C_{\text{total,II}}$, will be less than that without monitoring, $C_{\text{total,I}}$, and there will be the benefit of using SHM (i.e., $C_{\text{ben}} = C_{\text{total,I}} - C_{\text{total,II}} > 0$). On the contrary, if the monitoring is not cost-effective the minimum total life-cycle cost, $C_{\text{total,III}}$, will be larger than $C_{\text{total,I}}$ (see Figure 10.4a and c) and, as a result, there is a financial loss if SHM is used (i.e., $C_{\text{ben}} = C_{\text{total,I}} - C_{\text{total,III}} < 0$). To maximize the benefit of SHM, (1) optimum SHM installation (Worden and Burrows 2001; Meo and Zumpano 2005), (2) optimum monitoring scheduling (Kim and Frangopol 2010, 2011a, 2011b, 2011c), and (3) optimum use of SHM data (Frangopol et al. 2008a, 2008b) have to be all implemented.

10.3.2 Probabilistic Approaches for Bridge Performance Assessment and Prediction

In general, uncertainty can be divided into two types: aleatoric and epistemic. Aleatoric uncertainty is related to the inherent randomness of a process, and epistemic uncertainty arises from imperfect knowledge (Ang and Tang 2007). Both aleatoric and epistemic uncertainties have to be considered in the assessment and prediction of structural performance. To quantify the uncertainties in rational ways, probabilistic concepts and methods have to be used.

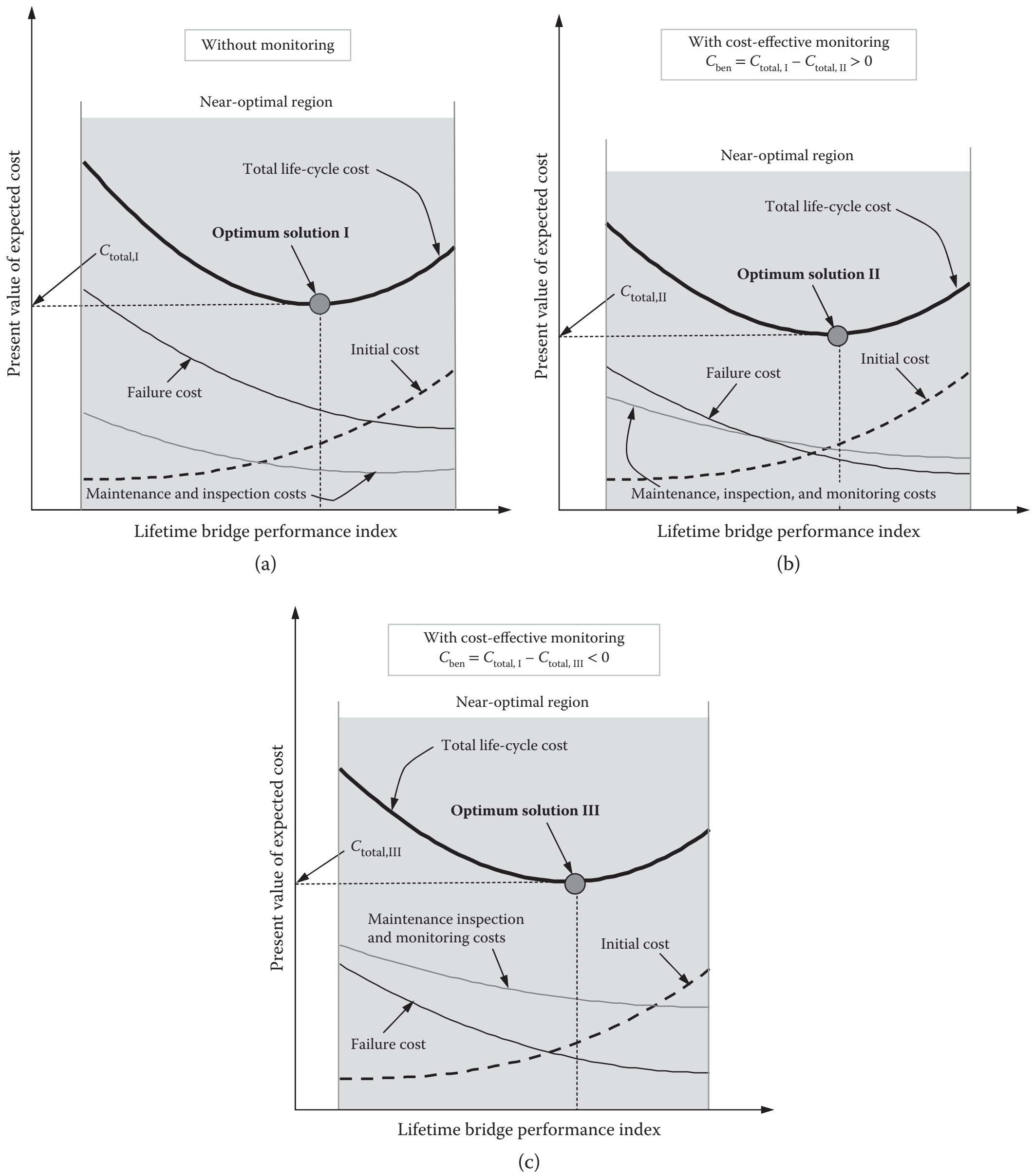

FIGURE 10.4 Relation between total life-cycle cost and lifetime bridge performance index in optimal region: (a) without monitoring, (b) with cost-effective monitoring, and (c) without cost-effective monitoring. (Adapted from Frangopol, D.M., *Struct. Infrastruct. E.*, 7(6), 389–413, 2011.)

10.3.2.1 Reliability

Reliability can be defined as the probabilistic measure of assurance of performance (Ang and Tang 1984). The reliability of an engineering system can be formulated in terms of safety margin M (i.e., difference between resistance R and load effect S). If R and S are random variables and the PDF of the margin of safety $M = R - S$ is available, the structural reliability (i.e., probability of survival), p_s, can be formulated as follows:

$$p_s = \int_0^{\infty} f_M(m)\,\mathrm{d}m \tag{10.3}$$

where $f_M(m)$ is PDF of the margin of safety M. Using the probability of survival, p_s, or the mean, μ_M, and standard deviation, σ_M, of the safety margin, reliability index β is defined as follows:

$$\beta = \Phi^{-1}(p_s) = \frac{\mu_M}{\sigma_M} \tag{10.4}$$

where $\Phi^{-1}(\cdot)$ is the inverse of the standard normal cumulative distribution function (CDF). As an example, consider independent, normally distributed resistance R and load effect S. The mean and standard deviation of R are assumed to be 250 and 20 MPa [i.e., $N(250; 20)$, where N indicates the normal distribution], respectively. The load effect S with and without updating based on monitoring data are associated with the normal distributions $N(150; 30)$ and $N(150; 50)$, respectively, as shown in Figure 10.5a. Figure 10.5b shows the PDFs of safety margin M with and without updating. According to Equation 10.3, p_s is the area under PDF of M above 0. The probabilities of survival p_s with and without updating based on monitoring are obtained as 0.9972 and 0.9683, respectively. Further details for the structural reliability associated with correlated and/or nonnormal random variables can be found in Ang and Tang (1984).

10.3.2.2 Statistics of Extremes

Bridge monitoring requires a large storage system if all monitored data are recorded and saved. The statistics of extremes is a well-suited concept for efficient monitored data management and bridge performance assessment and prediction (Frangopol et al. 2008b; Frangopol 2011; Messervey et al. 2011). Under the assumption that $Y_{\max}$ is the largest value among n samples taken from the population of an initial variate X (i.e., $Y_{\max} = \max\{X_1, X_2, \ldots, X_n\}$), the CDFs of the n initial variables are identical [i.e., $F_X(x) = F_{X1}(x) = F_{X2}(x) = \ldots = F_{Xn}(x)$] and these initial variables are statistically independent; the CDF of $Y_{\max}$ is obtained as follows:

$$F_{Y\max}(y) \equiv P(Y_{\max} \le y) = P(X_1 \le y, X_2 \le y, \ldots, X_n \le y) = \left[F_X(y)\right]^n \tag{10.5}$$

As the sample size n approaches infinity, $F_{Y\max}(y)$ may converge to an asymptotic distribution. Gumbel (1958) categorized the asymptotic distributions into (1) type I asymptotic form (i.e., the double exponential form), (2) type II asymptotic form (i.e., the exponential form), and (3) type III asymptotic form (i.e., the exponential form with upper bound). For example, if the initial variables are normally distributed, the tail of the initial distribution exponentially decays in the direction of the largest value and, therefore, the largest value is associated with type I asymptotic distribution. The CDF of the type I asymptotic form is (Ang and Tang 1984):

$$F_{Y\max}(y) = \exp\left[-\exp\left(-\upsilon_{\max}(y - \delta_{\max})\right)\right] \tag{10.6}$$

where $\delta_{\max}$ and $\upsilon_{\max}$ are the location and scale parameters, respectively. The location parameter $\delta_{\max}$ is defined as $F_X^{-1}(1 - 1/n)$, where $F_X^{-1}(\cdot)$ is inverse CDF of the initial variate X and n is sample size of the initial population X. The scale parameter $\upsilon_{\max}$ is $n \cdot f_X(\delta_{\max})$, where $f_X(\cdot)$ is PDF of the initial variate X. The exact and asymptotic CDFs of the largest value $Y_{\max}$ from a standard normal distribution are shown in Figure 10.6. These CDFs are obtained using Equation 10.5 and Equation 10.6, respectively. This figure shows that the CDF of $Y_{\max}$ converges to the CDF of the type I asymptotic form, as the sample size $n \to \infty$. Additional details on statistics of extremes including characteristics of the asymptotic forms for largest and smallest values and applications to fundamental engineering problems are available in Gumbel (1958) and Ang and Tang (1984).

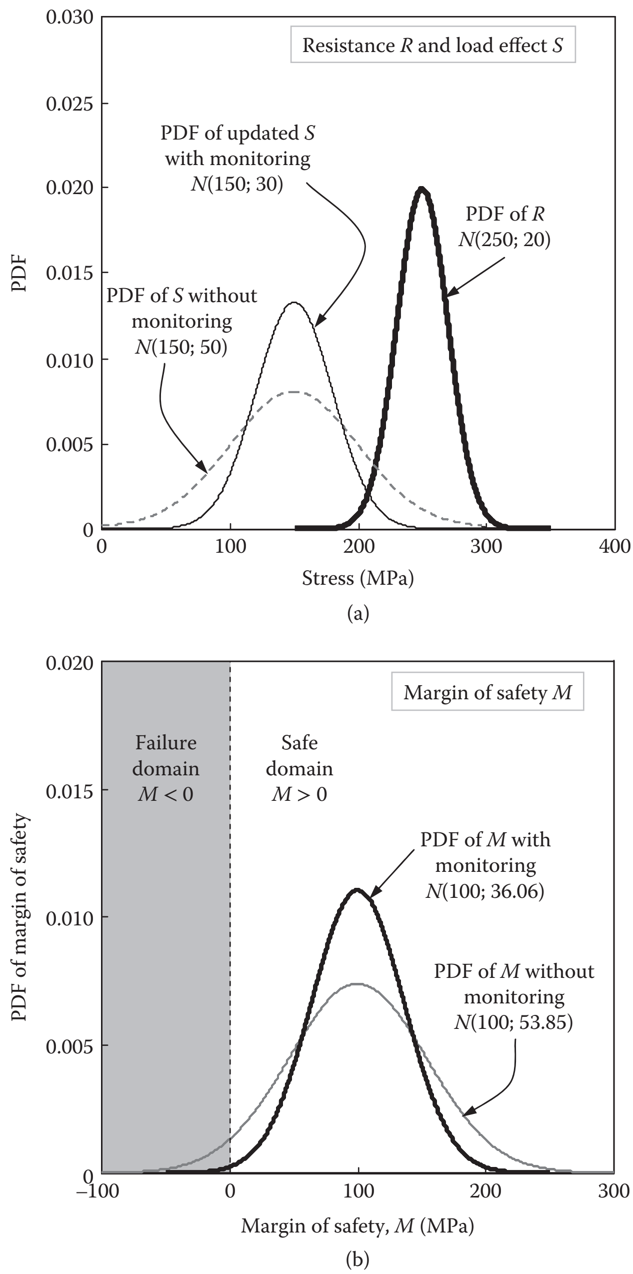

FIGURE 10.5 Possible effect of monitoring on (a) probability density functions (PDFs) of resistance R and load effect S and (b) PDFs of margin of safety M.

10.3.2.3 Exceedance Probability

Exceedance probability is a useful performance indicator. It represents the probability that in a predefined future time period the safety threshold of a physical quantity will be violated; for example, the probability that the effects of loadings due to very heavy trucks and extreme actions (e.g., flood, earthquake, and hurricane) will exceed a given probability threshold associated with a bridge in the next

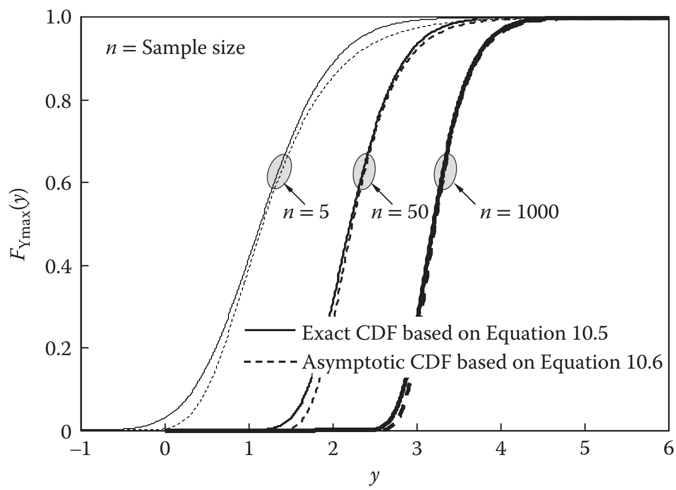

FIGURE 10.6 Exact and asymptotic cumulative distribution functions of the largest value from standard normal distribution.

year (Lambert et al. 1994; Kunreuther 2002; Liu et al. 2009a, 2009b). If $Y_{\max,n}$ is the largest value among n existing observations of the initial population X, and the initial variate X is associated with type I or type II asymptotic distribution, the probability that the largest value among N future observations, $Y_{\max,N}$, is greater than the largest value among n existing observations, $Y_{\max,n}$, is as follows:

$$P_{\text{exc}}(N,n) = P(Y_{\max,N} > Y_{\max,n}) = 1 - \exp[-N/n] \tag{10.7}$$

The exceedance probability will be used in Section 10.4.1.1 to define the availability of monitoring data for performance prediction and to find the optimum bridge monitoring plan.

10.3.3 Bridge Performance Assessment and Prediction Based on Monitoring Data

A probabilistic approach for efficient use of monitoring data has been proposed by Liu et al. (2009a). In this approach, the state function to estimate bridge performance is formulated considering monitored data, a time-variant function, which is used to predict a bridge component performance, and measurement error in the monitored data. The formulation of the time-variant function is based on statistics and probabilistic concepts (e.g., statistics of extremes and exceedance probability presented in Sections 10.3.2.2 and 10.3.2.3). Additional details including applications to the bridge components and systems can be found in the works by Liu et al. (2009a, 2009b).

The approach proposed by Liu et al. (2009a) was extended to multiple limit states including fatigue, serviceability, and ultimate structural capacity in the work by Orcesi et al. (2010) and Orcesi and Frangopol (2010b, 2011), where monitored data of an existing bridge (i.e., I-39 Northbound Wisconsin River Bridge, as shown in Figure 10.7) in Wisconsin is used. The monitoring program was conducted from July 2004 to November 2004 by the ATLSS Engineering Research Center at Lehigh University (Mahmoud et al. 2005). For reliability assessment and prediction, the state function for each limit state was formulated and the random variables of the state function were defined using the monitoring data. Figure 10.8 shows the reliability profiles of cross-sections 1 and 2 shown in Figure 10.7. The reliability profiles for fatigue in cross-section 1, serviceability in cross-section 2, and ultimate structural capacity of cross-section 2 are presented in Figure 10.8a, b, and c, respectively. Using these reliability profiles, the optimal maintenance strategies when multiple limit states are considered independently or simultaneously can be determined through optimization by minimizing failure cost and/or maintenance cost (Orcesi et al. 2010; Orcesi and Frangopol 2010a).

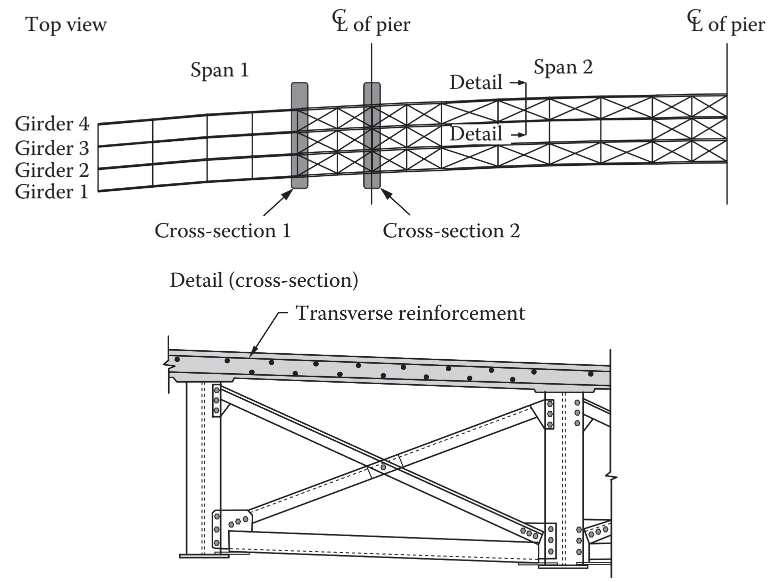

FIGURE 10.7 Top view and cross-sectional detail of the I-39 Northbound Wisconsin River Bridge. (Adapted from Mahmoud, H.N. et al., *ATLSS Report No. 05-04*, Lehigh University, Bethlehem, Pennsylvania, 2005.)

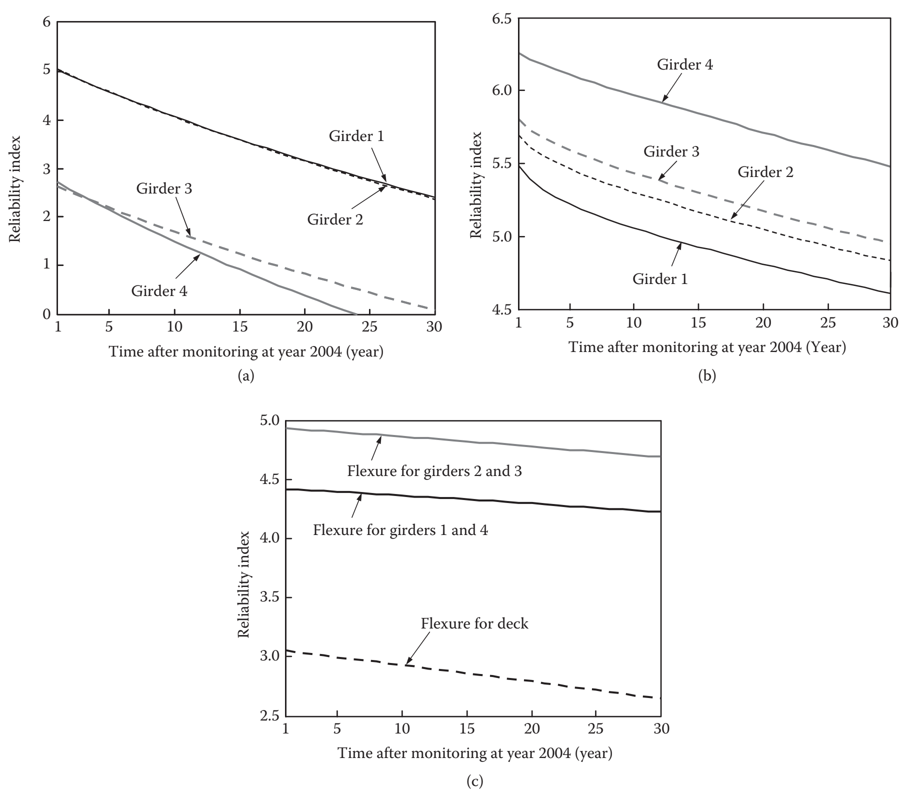

FIGURE 10.8 Reliability index profiles of Wisconsin River Bridge shown in Figure 10.7: (a) fatigue in cross-section 1, (b) serviceability in cross-section 2, and (c) ultimate structural capacity in cross-section 2. (Adapted from Orcesi, A.D. et al., *Eng. Struct.*, 32(3), 627–640, 2010.)

10.4 Integration of Bridge Health Monitoring in Life-Cycle Analysis

One of the main purposes of BHM is providing the database for optimum bridge life-cycle maintenance intervention, which leads to improving the efficiency of bridge life-cycle maintenance intervention, as mentioned in Section 10.2.4. To maximize the benefit of SHM within a life-cycle context, cost-effective monitoring planning should be considered (Kim and Frangopol 2011a, 2011c). Furthermore, lifetime optimization of monitoring, and maintenance strategies, needs to be investigated in a life-cycle management framework (Frangopol and Messervey 2009a, 2009b; Frangopol 2011).

10.4.1 Optimum Bridge Monitoring Planning

BHM is performed at uniform or nonuniform time intervals. For a monitoring plan with a uniform time interval, a biobjective optimization consisting of simultaneously minimizing the monitoring cost and maximizing the expected average availability of the monitoring data for performance prediction is solved. Nonuniform time interval monitoring planning is based on the expected damage detection delay (Kim and Frangopol 2012).

10.4.1.1 Monitoring Planning with Uniform Time Interval Based on Availability

The availability of monitoring data for performance prediction is defined as the probability that the available data can be used during prediction duration (Kim and Frangopol 2010). The formulation of availability of monitoring data is based on exceedance probability. As a result, the average availability $\bar{A}$ of monitoring data for given prediction and monitoring durations can be defined as follows (Ang and Tang 1984):

$$\bar{A} = \frac{T_L}{t_{\mathrm{pd}}} \cdot P_{\mathrm{exc}}\left(t_{\mathrm{pd}}, t_{\mathrm{mon}}\right) + \left(1 - P_{\mathrm{exc}}\left(t_{\mathrm{pd}}, t_{\mathrm{mon}}\right)\right) \quad \text{for } T_{\mathrm{L}} \le t_{\mathrm{pd}} \tag{10.8}$$

where T_{L} is the time to lose the usability of monitoring data (years); P_{exc} $(t_{\mathrm{pd}}, t_{\mathrm{mon}})$ is the exceedance probability, defined as $1 - \exp(-t_{\mathrm{pd}}/t_{\mathrm{mon}})$ (see Equation 10.7); and t_{pd} and t_{mon} are the prediction and monitoring durations in years, respectively. The expected average availability of monitoring data for prediction, $E(\bar{A})$, is expressed as follows (Ang and Tang 1984):

$$E\left(\bar{A}\right) = 1 - \frac{1}{t_{\mathrm{pd}}} \cdot \int_0^{t_{\mathrm{pd}}} P_{\mathrm{exc}}\left(t, t_{\mathrm{mon}}\right) \mathrm{d}t = \frac{t_{\mathrm{mon}}}{t_{\mathrm{pd}}}\left[1 - \exp\left(\frac{t_{\mathrm{pd}}}{t_{\mathrm{mon}}}\right)\right] \tag{10.9}$$

The total monitoring cost, C_{mon}, for m monitoring actions is estimated as follows:

$$C_{\mathrm{mon}} = C_{m,\mathrm{instl}} + m \cdot t_{\mathrm{mon}} \cdot C_{m,\mathrm{annual}} \tag{10.10}$$

where $C_{m,\mathrm{instl}}$ is initial cost of the monitoring system and $C_{m,\mathrm{annual}}$ is annual cost related to the operation, inspection, and repair of the monitoring system. In this chapter, both $C_{m,\mathrm{instl}}$ and $C_{m,\mathrm{annual}}$ are assumed to be 10.

The increase in monitoring duration is necessary to obtain sufficient monitoring data, leading to more reliable bridge performance prediction. However, as the monitoring duration increases, additional financial resources are required. The decision for optimal monitoring planning with uniform time interval is made through the formulation of a biobjective optimization considering both maximization of the expected average availability and minimization of the total monitoring cost. As an example, the cross-section 1 of the I-39 Northbound Wisconsin River Bridge shown in Figure 10.7 is considered. The biobjective optimization for cost-effective monitoring planning with uniform time interval is formulated as follows:

$$\text{Find } t_{\mathrm{mon}} \text{ and } t_{\mathrm{pd}} \tag{10.11a}$$

$$\text{to maximize } E\left(\bar{A}\right) \text{ and minimize } C_{\mathrm{mon}} \tag{10.11b}$$

$$\text{such that } 0.1 \text{ year} \leq t_{mon} \leq 1 \text{ year} \tag{10.11c}$$

$$0.1 \text{ year} \leq t_{pd} \leq 1 \text{ year} \tag{10.11d}$$

$$\text{given } m = 5 \tag{10.11e}$$

In the biobjective optimization, the design variables are the monitoring and prediction durations t_{mon} and t_{pd}, respectively, as indicated in Equation 10.11a. The objectives are to maximize the expected average availability, $E(\bar{A})$ (see Equation 10.9), and to minimize the total monitoring cost, C_{mon} (see Equation 10.10). The constraints are provided in Equations 10.11c and 10.11d. The number of monitoring actions m to estimate the total monitoring cost C_{mon} is assumed to be 5 (see Equation 10.11e). To find the Pareto optimal solution set for the biobjective optimization problem, the optimization toolbox (i.e., genetic algorithm for multiobjective optimization) provided in MATLAB® version R2011a (MathWorks Inc. 2011) is used in this chapter. Figure 10.9a shows a Pareto set of 100 solutions, and the optimal monitoring plans for solutions A1, A2, and A3 are illustrated in Figure 10.9b. If solution A1 is selected, monitoring duration t_{mon} and prediction duration t_{pd} are 0.24 and 0.19 years, respectively (see Figure 10.9b). The values of the expected average availabilities of solutions A1, A2, and A3 are 0.7, 0.8, and 0.9, respectively, and their associated total monitoring costs are 22.09, 30.55, and 54.72, respectively.

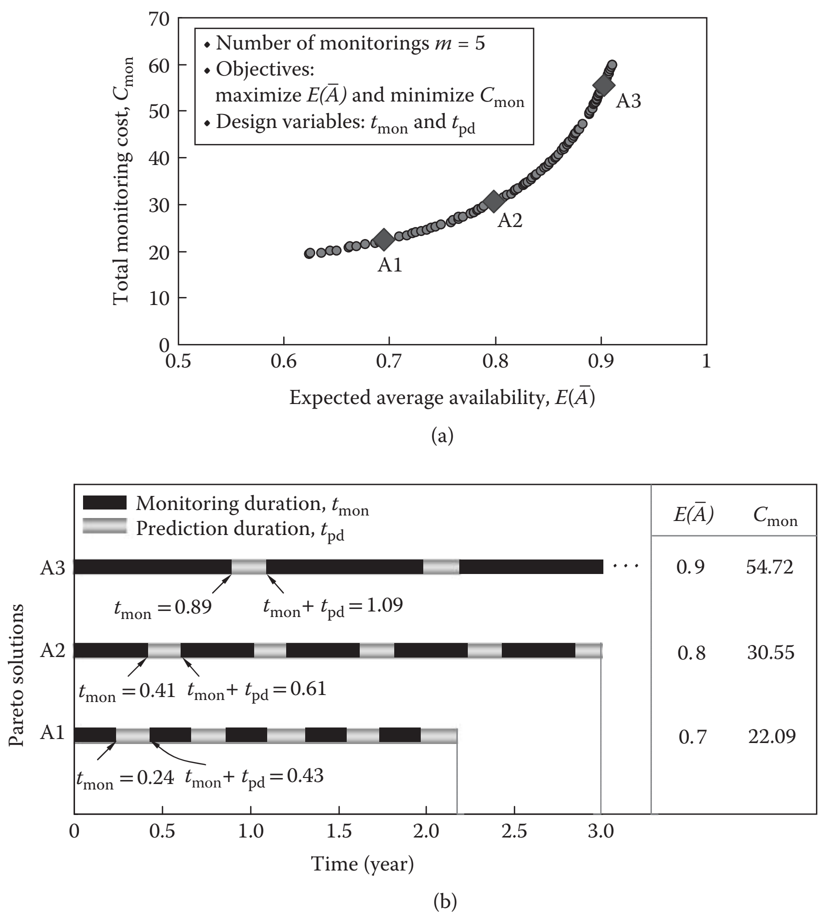

FIGURE 10.9 (a) Pareto solution set and (b) optimal monitoring plans with uniform time interval for solutions A1, A2, and A3 in (a).

10.4.1.2 Monitoring Planning with a Nonuniform Time Interval Based on Damage Detection Delay

An effective monitoring plan with a nonuniform time interval is based on damage detection delay while taking into account damage occurrence and propagation under uncertainty. Damage detection delay is defined as the time lapse since the structure has been damaged until the damage is detected by monitoring (Huang and Chiu 1995). Under the assumptions that the probability of detection during monitoring t_{mon} is perfect and the two monitoring actions are applied as shown in Figure 10.10, the expected damage detection delay, $E(t_{del})$, becomes as follows:

$$E(t_{del}) = t_{ms,1} - t \quad \text{for } t \le t_{ms,1} \tag{10.12a}$$

$$E(t_{del}) = t_{ms,2} - t \quad \text{for } t_{ms,1} + t_{mon} \le t \le t_{ms,2} \tag{10.12b}$$

where t is the damage occurrence time, $t_{ms,i}$ is the ith monitoring starting time, and t_{mon} is the monitoring duration. During the time intervals $t_{ms,1} \le t \le t_{ms,1} + t_{mon}$ and $t_{ms,2} \le t \le t_{ms,2} + t_{mon}$ there is no damage detection delay, since it is assumed that the probability of damage detection during monitoring is perfect. Considering the PDF of damage occurrence time $f_T(t)$ (see Figure 10.10), the expected damage detection delay based on Equation 10.12 is obtained as follows:

$$E(t_{del}) = \int_{t_{lb}}^{t_{ms,1}} (t_{ms,1} - t) f_T(t)\,dt + \int_{t_{ms,1}+t_{mon}}^{t_{ms,2}} (t_{ms,2} - t) f_T(t)\,dt \tag{10.13}$$

where t_{lb} and $t_{ms,2} + t_{mon}$ are the lower and upper bounds of damage occurrence, respectively. It is assumed that (1) there is no damage before the lower bound t_{lb} and (2) near the time $t_{ms,2} + t_{mon}$ (*i.e.*, upper bound, t_{ub}) the probability of damage occurrence is very high. If a monitoring action is applied m times with the same duration t_{mon}, the expected damage detection delay, $E(t_{del})$, is expressed as follows (Kim and Frangopol 2011b):

$$E(t_{del}) = \sum_{i=1}^{m} \left[\int_{t_{ms,i-1}+t_{mon}}^{t_{ms,i}} (t_{ms,i} - t) f_T(t)\,dt \right] \tag{10.14}$$

where $t_{ms,0} + t_{mon} = t_{lb}$ for $i = 1$ and $t_{ms,n} + t_{mon} = t_{ub}$ for $i = m$ are the lower and upper bounds of damage occurrence time, respectively. In this chapter, t_{lb} and t_{ub} are defined as follows:

$$t_{lb} = F_T^{-1}(\Phi(-3)) \tag{10.15a}$$

$$t_{ub} = F_T^{-1}(\Phi(3)) \tag{10.15b}$$

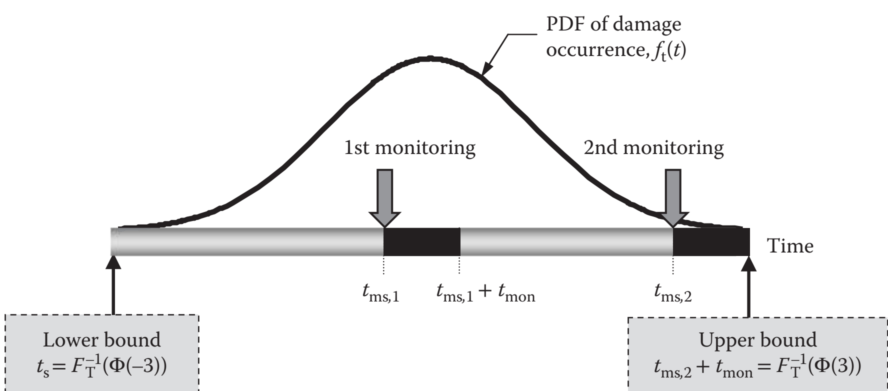

FIGURE 10.10 Probability density function (PDF) of damage occurrence, and lower and upper bounds of damage occurrence time.

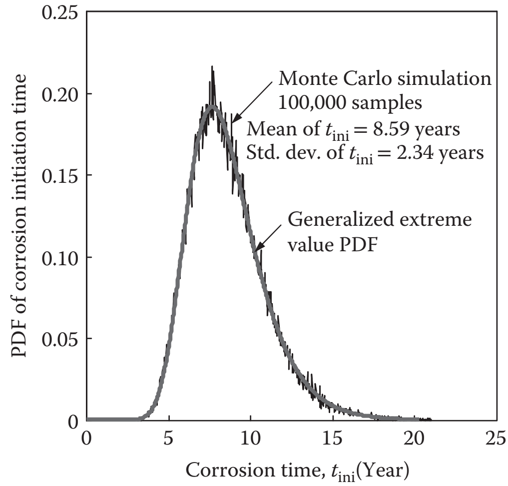

FIGURE 10.11 Probability density function (PDF) of corrosion initiation time of reinforcement of the concrete slab.

where $F_T^{-1}(\cdot)$ is the inverse of the CDF of damage occurrence time t and $\Phi(\cdot)$ is the standard normal CDF.

As an illustrative example, consider corrosion of reinforcement in the concrete slab of I-39 Northbound Bridge (see Figure 10.7). Corrosion initiation serves as the damage criterion. Using Fick's second law, the PDF of corrosion initiation time, t_{ini} (i.e., damage occurrence time), of the slab is estimated as shown in Figure 10.11. To apply timely maintenance actions, damage detection delay should be minimized. Increasing the number of monitoring actions and/or monitoring duration can lead to a reduction of expected damage detection delay. However, this requires additional financial resources. Therefore, cost-effective monitoring planning should be considered as the solution of a biobjective optimization, simultaneously minimizing both the expected damage detection delay and the total monitoring cost. The biobjective optimization problem for monitoring planning is formulated as follows:

$$\text{Find } \boldsymbol{t}_{\mathbf{ms}} = \left\{t_{\text{ms},1}, t_{\text{ms},2}, \ldots, t_{\text{ms},m}\right\} \text{ and } t_{\text{mor}} \quad (10.16a)$$

$$\text{to minimize both } E\left(t_{\text{del}}\right) \text{ and } C_{\text{mon}} \quad (10.16b)$$

$$\text{such that } t_{\text{ms},i} - t_{\text{ms},i-1} \geq 1 \text{ year} \quad (10.16c)$$

$$0.1 \text{ year} \leq t_{\text{mon}} \leq 1.0 \text{ year} \quad (10.16d)$$

$$\text{given } f_T(t), m = 5 \quad (10.16e)$$

In this biobjective optimization problem, the objectives are minimization of both the expected damage detection delay $E(t_{del})$ (see Equation 10.14) and the total monitoring cost C_{mon} (see Equation 10.10). The design variables are the vector of monitoring starting times t_{ms} and monitoring duration t_{mon}, as indicated in Equation 10.16a. The time interval between monitoring starting times should be at least 1 year, and the monitoring duration has to be in the interval 0.1–1.0 year, as indicated in Equations 10.16c and 10.16d. The PDF of damage occurrence (i.e., PDF of corrosion initiation time) in Figure 10.11 and number of monitorings $m = 5$ are given (see Equation 10.16e). Through the genetic algorithm process with 200 generations, a Pareto set of 100 solutions are obtained, as shown in Figure 10.12a. Figure 10.12b presents the monitoring plans of three solutions B1, B2, and B3 in Figure 10.12a. For solution B1 in Figure 10.12a, the associated $E(t_{del})$ and C_{mon} are 0.3 years and 55.23, respectively, as indicated in Figure 10.12b. The monitoring plan for solution B1 requires a monitoring duration of 0.9 years. If solution B3 instead of solution B1 is selected as a monitoring plan, $E(t_{del})$ will increase from 0.3 to 0.7 years but C_{mon} will be reduced from 55.23 to 23.57.

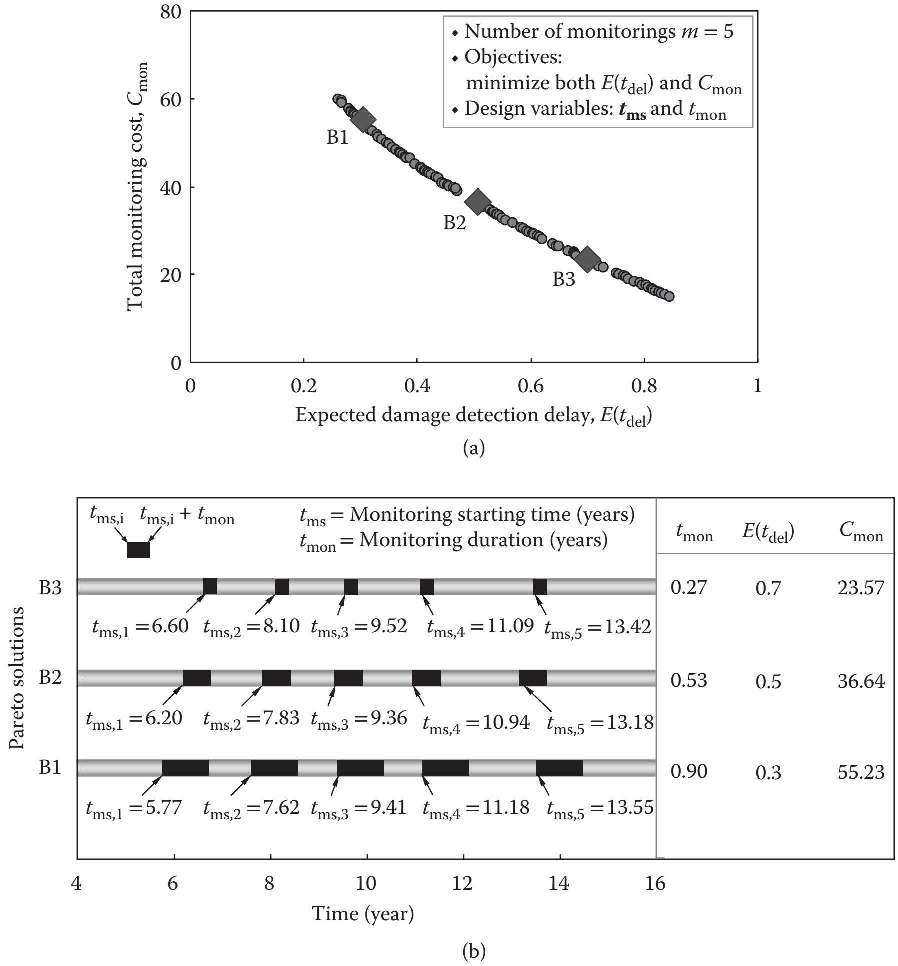

FIGURE 10.12 (a) Pareto solution set and (b) optimal monitoring plans with nonuniform time interval for solutions B1, B2, and B3 in (a).

10.4.2 Integrated Life-Cycle Bridge Health Monitoring, Maintenance, and Management

Optimum bridge life-cycle maintenance management is highly affected by the prediction model for bridge performance under uncertainty. During the last two decades, several significant studies have been conducted to develop a probabilistic approach for optimum bridge life-cycle management (Mori and Ellingwood 1994; Frangopol et al. 1997; Enright and Frangopol 1999; Estes and Frangopol 1999; Frangopol et al. 2001; Kong and Frangopol 2003, 2005; Liu and Frangopol 2005; Liu et al. 2011, Bocchini and Frangopol 2011). Figure 10.13 presents the schematic for probabilistic bridge life-cycle analysis. Based on the lifetime performance prediction of a deteriorating bridge using probabilistic concepts and methods, optimization with an objective such as maximizing bridge lifetime performance or minimizing the bridge's expected total life-cycle cost is conducted. As a result, the expected structural performance, total life-cycle cost, and optimum inspection, monitoring, maintenance, and repair times can be obtained, as indicated in Figure 10.13.

Recently, SHM providing valuable information about bridge performance has been treated as an attractive tool in life-cycle performance and cost analysis, as well as in the diagnosis and prognosis of

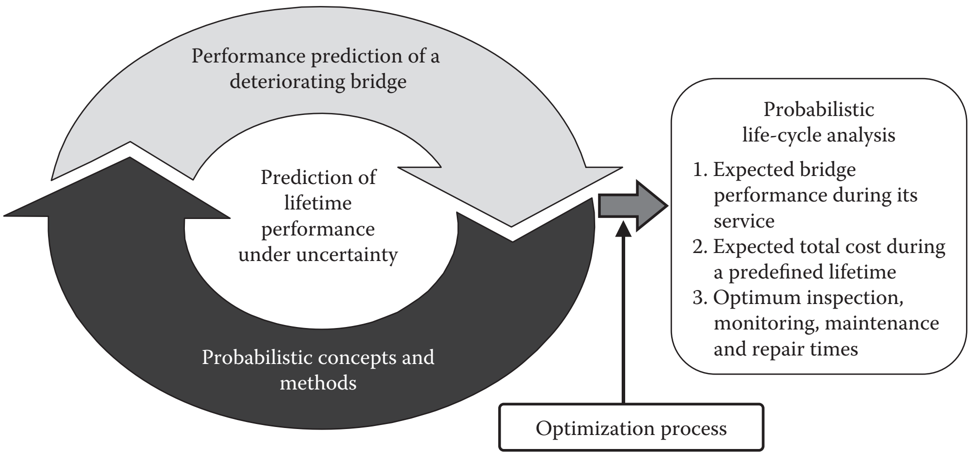

FIGURE 10.13 Schematic for probabilistic bridge life-cycle analysis.

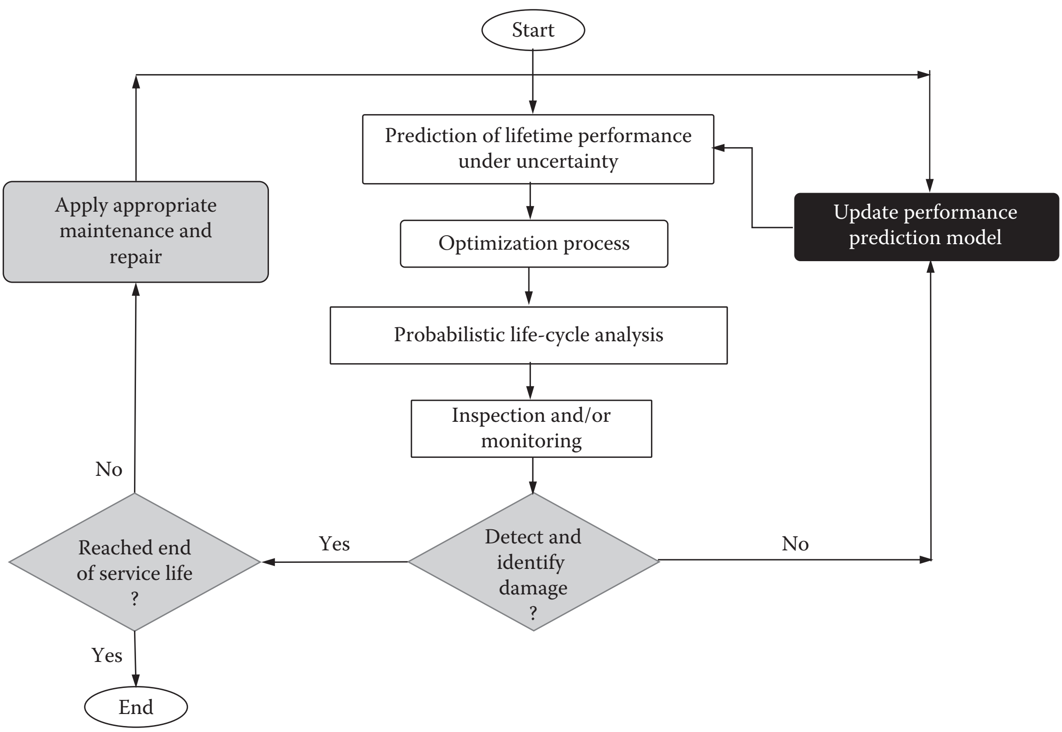

FIGURE 10.14 Flowchart of integrated life-cycle bridge inspection, health monitoring, maintenance, and management.

overall structural integrity (Farrar and Lieven 2007; Liu et al. 2009a, 2009b). When BHM does not detect damage, monitoring provides additional data, which can be used to improve the accuracy of bridge performance assessment and prediction by updating the existing prediction model. When damage is detected, identified, and repaired, life-cycle analysis for the repaired bridge system is required since the damage may reoccur and the performance of the bridge degrades over time. Figure 10.14 shows a flowchart of integrated life-cycle bridge inspection, health monitoring, maintenance, and management.

Life-cycle cost and performance analysis and their respective updates as shown in this figure are repetitive but necessary. The detailed computational platform for integrated life-cycle management of highway bridges including the updating procedure based on monitoring data can be found in the work by Okasha and Frangopol (2012).

10.5 Conclusions

BHM has been applied to a wide range of civil structures, including bridges, for various purposes. These purposes are related to the reduction of uncertainty associated with diagnosis and prognosis of structural performance. This chapter presented (1) an overview of the effect of BHM on bridge performance management under uncertainty, (2) probabilistic approaches for efficient use of monitoring data and cost-effective monitoring planning, and (3) a concept of integrated life-cycle framework using bridge monitoring for improved bridge management under uncertainty. Further efforts should be devoted to developing a generalized and integrated probabilistic procedure for optimum BHM under conflicting objectives such as monitoring cost and accuracy of bridge performance assessment and prediction.

Acknowledgments

The support by grants from the National Science Foundation award CMS-0639428; the Commonwealth of Pennsylvania, Department of Community and Economic Development, through the Pennsylvania Infrastructure Technology Alliance; the U.S. Federal Highway Administration Cooperative Agreement award DTFH61-07-H-00040; the U.S. Office of Naval Research award numbers N00014-08-1-0188 and N00014-12-1-0023; and the National Aeronautics and Space Administration award NNX10AJ20G is gratefully acknowledged. The authors acknowledge the comments offered by Alysson Mondoro, PhD student, Lehigh University. The opinions and conclusions presented in this paper are those of the authors and do not necessarily reflect the views of the sponsoring organizations.

Notation

$\bar{A}$ = average availability
C_{ben} = benefit of SHM
C_{fail} = expected cost of failure
C_{ini} = initial design and construction cost
C_{insp} = expected cost of inspection
$C_{m,\mathrm{annual}}$ = annual cost related to operation, inspection, and repair of the monitoring system
$C_{m,\mathrm{instl}}$ = initial cost of the monitoring system
C_{main} = expected cost of maintenance
C_{mon} = monitoring cost
C_{total} = expected total life-cycle cost
$E(X)$ = expected value of random variable X
$F_X(x)$ = CDF of random variable X
$F_X^{-1}(x)$ = inverse CDF of random variable X
$f_X(x)$ = PDF of random variable X
P_{exc} = exceedance probability
$\mathrm{PI}_{\mathrm{ini}}$ = initial bridge performance index
PI_t = bridge performance index at time t
$\mathrm{PI}_{\mathrm{thres}}$ = predefined threshold of performance index
p_{s} = probability of survival
R = resistance
r_{p} = degradation rate of structural performance

S = loading effect
T_L = time to lose usability of monitoring data
t_{del} = damage detection delay
t_{ini} = damage initiation time
t_{lb} = lower bound of damage occurrence
t_{life} = service life
t_{mon} = monitoring duration
t_{ms} = monitoring starting time
t_{pd} = prediction duration
t_{ub} = upper bound of damage occurrence
β = reliability index
μ_X = mean of random variable X
σ_X = standard deviation of random variable X

References

Ang, A.H.-S. and Tang, W.H. 1984. *Probability Concepts in Engineering Planning and Design Volume II*. John Wiley & Sons, New York.

Ang, A.H.-S. and Tang, W.H. 2007. *Probability Concepts in Engineering: Emphasis on Applications to Civil and Environmental Engineering*. 2nd edn. John Wiley & Sons, New York.

Bocchini, P. and Frangopol, D.M. 2011. "A probabilistic computational framework for bridge network optimal maintenance scheduling," *Reliability Engineering & System Safety*, Elsevier, 96(2), 332–349.

Brownjohn, J.M.W. 2007. "Structural health monitoring of civil infrastructure." *Philosophical Transactions of the Royal Society A*, Royal Society Publishing, 365(1851), 589–622.

Catbas, F.N., Susoy, M., and Frangopol, D.M. 2008. "Structural health monitoring and reliability estimation: Long span truss bridge application with environmental monitoring data," *Engineering Structures*, Elsevier, 30(9), 2347–2359.

Catbas, N., Gokce, H.B., and Frangopol, D.M. 2013a. "Predictive analysis by incorporating uncertainty through a family of models calibrated with structural health-monitoring data," *Journal of Engineering Mechanics*, ASCE, 139(6), 712–723.

Catbas, F.N., Gul, M., Gokce, H. B., Zaurin, R., Frangopol, D.M., and Grimmelsman, K. A. 2013b. "Critical issues, condition assessment and monitoring of heavy movable structures: Emphasis on movable bridges," *Structure and Infrastructure Engineering*, Taylor & Francis, doi:10.1080/15732479.2012.744060 (in press).

Cheung, M.S. and Li, W.C. 2003. "Probabilistic fatigue and fracture analyses of steel bridges." *Structural Safety*, Elsevier, 25(3), 245–262.

Connor, R.J. and Fisher, J.W. 2002. "Report on field inspection, assessment, and analysis of floor-beam connection cracking on the Birmingham Bridge." *ATLSS Report No. 02-10*, Lehigh University, Bethlehem, PA.

Connor, R.J. Fisher, J.W., Hodgson, I.C., and Bowman, C.A. 2004. "Results of field monitoring prototype floor-beam connection retrofit details on the Birmingham Bridge. Final Report." *ATLSS Report No. 04-04*, Lehigh University, Bethlehem, PA.

Connor, R.J. and McCarthy, J. R. 2006. "Report on field measurements and uncontrolled load testing of the Lehigh River Bridge (SR-33). Phase II. Final Report." *ATLSS Report No. 06-12*, Lehigh University, Bethlehem, PA.

Connor, R.J. and Santosuosso, B.J. 2002. "Report on field measurements and controlled load testing of the Lehigh River Bridge (SR-33). Final Report." *ATLSS Report No. 02-07*, Lehigh University, Bethlehem, PA.

Enright, M.P. and Frangopol, D.M. 1998a. "Service-life prediction of deteriorating concrete bridges." *Journal of Structural Engineering*, ASCE, 124(3), 309–317.

Enright, M.P. and Frangopol, D.M. 1998b. "Probabilistic analysis of resistance degradation of reinforced concrete bridge beams under corrosion." *Engineering Structures*, Elsevier, 20(11), 960–971.

Enright, M.P. and Frangopol, D.M. 1999. "Maintenance planning for deteriorating concrete bridges." *Journal of Structural Engineering*, ASCE, 125(12), 1407–1414.

Estes, A.C. and Frangopol, D.M. 1999. "Repair optimization of highway bridges using system reliability approach." *Journal of Structural Engineering*, ASCE, 125(7), 766–775.

Farrar, C.R. and Lieven, N.A.J. 2007. "Damage prognosis: the future of structural health monitoring." *Philosophical Transactions of the Royal Society A*, Royal Society Publishing, 365(1851), 623–632.

Farrar, C.R. and Worden, K. 2007. "An introduction to structural health monitoring" *Philosophical Transactions of the Royal Society A*, Royal Society Publishing, 365(1851), 303–315.

Frangopol, D.M. 2011. "Life-cycle performance, management, and optimization of structural systems under uncertainty: accomplishments and challenges." *Structure and Infrastructure Engineering*, Taylor & Francis, 7(6), 389–413.

Frangopol, D.M. and Kim, S. 2011. "Service life, reliability and maintenance of civil structures." Chapter 5 in *Service Life Estimation and Extension of Civil Engineering Structures*, L.S. Lee and V. Karbari, eds., Woodhead Publishing Ltd., Cambridge, U.K., 145–178.

Frangopol, D.M., Kong, J.S., and Gharaibeh, E.S. 2001. "Reliability-based life-cycle management of highway bridges." *Journal of Computing in Civil Engineering*, ASCE, 15(1), 27–34.

Frangopol, D.M., Lin, K.Y., and Estes, A.C. 1997. "Reliability of reinforced concrete girders under corrosion attack." *Journal of Structural Engineering*, ASCE, 123(3), 286–297.

Frangopol, D.M. and Maute, K. 2003. "Life-cycle reliability-based optimization of civil and aerospace structures." *Computers and Structures*, Elsevier, 81(7), 397–410.

Frangopol, D.M. and Messervey, T.B. 2009a. "Life-cycle cost and performance prediction: role of structural health monitoring." Chapter 16 in *Frontier Technologies for Infrastructures Engineering*, S-S. Chen and A.H.S. Ang, eds., CRC Press-Balkema-Taylor & Francis Group, Leiden, The Netherlands, 361–381.

Frangopol, D.M. and Messervey, T.B. 2009b. "Maintenance principles for civil structures." Chapter 89 in *Encyclopedia of Structural Health Monitoring*, C. Boller, F.K. Chang, and Y. Fujino, eds., John Wiley & Sons Ltd, Chicester, U.K., Vol. 4, 1533–1562.

Frangopol, D.M. and Messervey, T.B. 2011. "Effect of monitoring on reliability of structures." Chapter 18 in *Monitoring Technologies for Bridge Management*, B. Bakht, A.A. Mufti, and L.D. Wegner, eds., Multi-Science Publishing Co. Ltd. U.K., 515–560.

Frangopol, D.M., Strauss, A., and Kim, S. 2008a. "Bridge reliability assessment based on monitoring." *Journal of Bridge Engineering*, ASCE, 13(3), 258–270.

Frangopol, D.M., Strauss, A., and Kim, S. 2008b. "Use of monitoring extreme data for the performance prediction of structures: General approach." *Engineering Structures*, Elsevier, 30(12), 3644–3653.

Gumbel, E.J. 1958. *Statistics of Extremes*. Columbia Univ. Press, New York, NY.

Hassanain, M.A. and Loov, R.E. 2003. "Cost optimization of concrete bridge infrastructure." *Canadian Journal of Civil Engineering*, NRC Research Press, 30(5), 841–849.

Huang, B.-S. and Chiu, H.-N. 1995. "The quality management of the imperfect production process under two monitoring policies." *International Journal of Quality & Reliability Management*, Emerald, 12(3), 19–31.

Kim, S. and Frangopol, D.M. 2010. "Optimal planning of structural performance monitoring based on reliability importance assessment." *Probabilistic Engineering Mechanics*, Elsevier, 25(1), 86–98.

Kim, S. and Frangopol, D.M. 2011a. "Cost-effective lifetime structural health monitoring based on availability." *Journal of Structural Engineering*, ASCE, 137(1), 22–33.

Kim, S. and Frangopol, D.M. 2011b. "Inspection and monitoring planning for RC structures based on minimization of expected damage detection delay," *Probabilistic Engineering Mechanics*, Elsevier, 26(2), 308–320.

Kim, S. and Frangopol, D.M. 2011c. "Cost-based optimum scheduling of inspection and monitoring for fatigue sensitive structures under uncertainty." *Journal of Structural Engineering*, ASCE, 137(11), 1319–1331.

Kim, S. and Frangopol, D.M. 2012. "Probabilistic bicriterion optimum inspection/monitoring planning: applications to naval ships and bridges under fatigue," *Structure and Infrastructure Engineering*, Taylor & Francis, 8(10), 912–927.

Ko, J.M. and Ni, Y.Q. 2005. "Technology developments in structural health monitoring of large-scale bridges." *Engineering Structures*, Elsevier, 27(12), 1715–1725.

Kong, J.S. and Frangopol, D.M. 2003. "Life-cycle reliability based maintenance cost optimization of deteriorating structures with emphasis on bridges." *Journal of Structural Engineering*, ASCE, 129(6), 818–828.

Kong, J.S. and Frangopol, D.M. 2005. "Probabilistic optimization of aging structures considering maintenance and failure cost." *Journal of Structural Engineering*, ASCE, 131(4), 600–616.

Kunreuther, H. 2002. "Risk analysis and risk management in an uncertain world." *Risk Analysis: An International Journal*, Society for Risk Analysis, 22(4), 655–664.

Kwon, K. and Frangopol, D.M. 2010. "Bridge fatigue reliability assessment using probability density functions of equivalent stress range based on field monitoring data." *International Journal of Fatigue*, Elsevier, 32(8), 1221–1232.

Lambert, J.H., Matalas, N.C., Ling, C.W., Haimes, Y.Y., and Li, D. 1994. "Selection of probability distributions in characterizing risk of extreme events." *Risk Analysis: An International Journal*, Society for Risk Analysis, 14(5), 731–742.

Li, C.Q., Lawanwisut, W., and Zheng, J.J. 2005. "Time-dependent reliability method to assess the serviceability of corrosion-affected concrete structures." *Journal of Structural Engineering*, ASCE, 131(11), 1674–1680.

Liu, M. and Frangopol, D.M. 2005. "Bridge annual maintenance prioritization under uncertainty by multiobjective combinatorial optimization." *Computer-aided Civil and Infrastructure Engineering*, 20(5), 343–353.

Liu, M., Frangopol, D.M., and Kim, S. 2009a. "Bridge safety evaluation based on monitored live load effects." *Journal of Bridge Engineering*, ASCE, 14(4), 257–269.

Liu, M., Frangopol, D.M., and Kim, S. 2009b. "Bridge system performance assessment from structural health monitoring: A case study." *Journal of Structural Engineering*, ASCE, 135(6), 733–742.

Liu, M., Frangopol, D.M., and Kwon, K. 2010. "Fatigue reliability assessment of retrofitted steel bridges integrating monitored data" *Structural Safety*, Elsevier, 32(1), 77–89.

Liu, Y., Li, Y., Huang, H., and Kuang, Y. 2011. "An optimal sequential preventive maintenance policy under stochastic maintenance quality." *Structure and Infrastructure Engineering*, Taylor & Francis, 7(4), 315–322.

Lukić, M. and Cremona, C. 2001. "Probabilistic assessment of welded joints versus fatigue and fracture." *Journal of Structural Engineering*, ASCE, 127(2), 211–218.

Mahmoud, H.N., Connor, R.J., and Bowman, C.A. 2005. "Results of the fatigue evaluation and field monitoring of the I-39 Northbound Bridge over the Wisconsin River." *ATLSS Report No. 05-04*, Lehigh University, Bethlehem, PA.

MathWorks Inc. 2011. *Optimization Toolbox™ 6 User's Guide*. The MathWorks, Inc., USA.

Meo, M. and Zumpano, G. 2005. "On the optimal sensor placement techniques for a bridge structure." *Engineering Structures*, Elsevier, 27(10), 1488–1497.

Messervey, T.B. 2008. "Integration of structural health monitoring into the design, assessment, and management of civil infrastructure." Ph.D. Dissertation, University of Pavia, Pavia, Italy.

Messervey, T.B., Frangopol, D.M., and Casciati, S. 2011. "Application of the statistics of extremes to the reliability assessment and performance prediction of monitored highway bridges." *Structure and Infrastructure Engineering*, Taylor & Francis, 7(1–2), 87–99.

Mori, Y. and Ellingwood, B.R. 1994. "Maintaining reliability of concrete structures. II: Optimum inspection/repair." *Journal of Structural Engineering*, ASCE, 120(3), 846–862.

Okasha, N.M. and Frangopol, D.M. 2012. "Integration of structural health monitoring in a system performance based life-cycle bridge management framework," *Structure and Infrastructure Engineering*, Taylor & Francis, 8(11), 999–1016.

Orcesi, A.D. and Frangopol, D.M. 2010a. "Optimization of bridge management under budget constraints: role of structural health monitoring." *Transportation Research Record: Journal of the Transportation Research Board* (Bridge Engineering 2010: Volumes 1–3), 3(2202), 148–158.

Orcesi, A.D. and Frangopol, D.M. 2010b. "Inclusion of crawl tests and long-term health monitoring in bridge serviceability analysis," *Journal of Bridge Engineering*, ASCE, 15(3), 312–326.

Orcesi, A.D. and Frangopol, D.M. 2011. "Probability-based multiple-criteria optimization of bridge maintenance using monitoring and expected error in the decision process," *Structural and Multidisciplinary Optimization*, Springer, 44(1), 137–148.

Orcesi, A.D. and Frangopol, D.M. 2013. "Bridge performance monitoring based on traffic data," *Journal of Engineering Mechanics*, ASCE (in press).

Orcesi, A.D., Frangopol, D.M., and Kim, S. 2010. "Optimization of bridge maintenance strategies based on multiple limit states and monitoring." *Engineering Structures*, Elsevier, 32(3), 627–640.

Righiniotis, T.D. and Chryssanthopoulos, M.K. 2003. "Probabilistic fatigue analysis under constant amplitude loading." *Journal of Constructive Steel Research*, Elsevier, 59(7), 867–886.

Stewart, M.G. 2004. "Spatial variability of pitting corrosion and its influence on structural fragility and reliability of RC beams in flexure." *Structural Safety*, Elsevier, 26(4), 453–470.

Strauss, A., Frangopol, D. M., and Kim, S. 2008. "Use of monitoring extreme data for the performance prediction of structures: Bayesian updating," Engineering Structures, Elsevier, 30(12), 3654–3666.

Worden, K. and Burrows, A.P. 2001. "Optimal sensor placement for fault detection." *Engineering Structures*, Elsevier, 23(8), 885–901.

Zhu, B. and Frangopol, D.M. 2013. "Incorporation of SHM data on load effects in the reliability and redundancy assessment of ships using Bayesian updating," *Structural Health Monitoring*, Sage, 12(4), 377–392.

11

Bridge Maintenance

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11.1 Introduction

Transportation infrastructure has always played a significant role in maintaining the quality of life of people and the economy of the countries around the world. In the past, users expected safe and secure transportation. Recent advances in technologies, such as the Internet and air transportation, have led to rapid globalization and the increased significance of our transportation infrastructure. This has also changed the expectations of transportation users as well as stakeholders. Uninterrupted mobility and reliability, besides safety and security, are now expected to maintain competitive advantage.

Bridges are an integral part of the transportation infrastructure and every aspect of their life cycle is extremely important to maintaining the stakeholders' expectations that include, but are not limited to, safety, security, mobility, and reliability. At the same time, the last decade has seen several personnel and financial resource constraints. Thus, the infrastructure has to be managed effectively, while maintaining the required safety, at optimal costs.

Maintenance is an integral part of bridge management in ensuring the safety and durability of bridges in a cost-effective manner. Given maintenance can be defined as an act to keep bridges in a specified state, it plays a major role in infrastructure management by keeping the bridges operational to achieve stakeholder's expectations. Appropriate maintenance extends bridge service life cost-effectively, thus avoiding unnecessary or premature rehabilitation or replacement of the structure or its components. An effective maintenance program involves several aspects, including type and scope of an activity, scheduling the activity, and determining costs associated with the activity. All these aspects have changed considerably in the last four decades from very reactive to very proactive in nature. This chapter briefly discusses the role of maintenance in a bridge's life cycle, maintenance activity types, and new evolving trends in effectively managing the bridges.

Major bridges, such as suspension bridges (see Figures 11.1 and 11.2), are complex structures, very long, generally span over large bodies of water, and carry much larger traffic volumes. These bridges are generally maintained by dedicated authorities or offices and thus are treated separately. Thus, this chapter is more geared toward general bridges maintained by state transportation agencies (Figures 11.3 through 11.5).

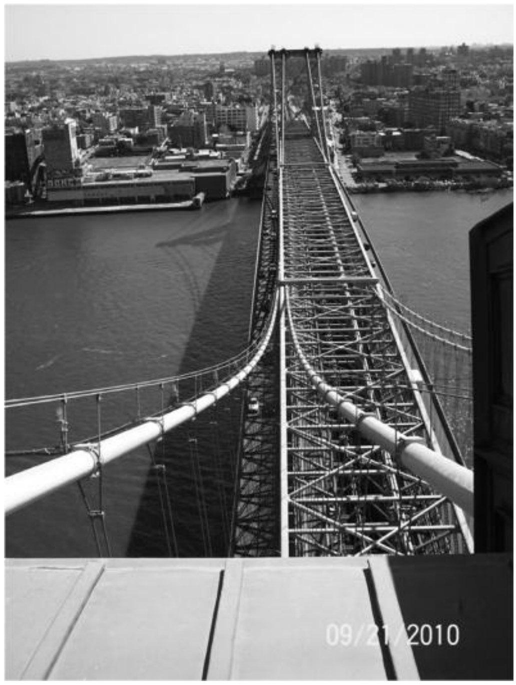


FIGURE 11.1 Suspension bridge.

FIGURE 11.2 Suspension bridge.

FIGURE 11.3 Through truss bridge. (Courtesy of the New York State Department of Transportation.)

FIGURE 11.4 Steel-girder bridge with concrete deck. (Courtesy of the New York State Department of Transportation.)

FIGURE 11.5 Concrete girder bridge. (Courtesy of the New York State Department of Transportation.)

11.2 Role of Maintenance in Bridge Life Cycle

The bridge cycle begins with planning based on the expected needs through stakeholder demands and/or network analysis. Once the need is identified, approximate location and overall expectations for the structure are set in the planning stage. Then the bridge is designed to meet the current and expected demands in its life cycle as well as site conditions based on existing codes. In most cases, life span and future demands are indirectly taken into consideration through the use of prevailing national and local design specifications, guidelines, and regulations. After the bridge is designed, the bridge is constructed based on the construction documents including design plans and construction specifications with changes made during the construction to accommodate unexpected issues resulting from fabrication, material transportation, weather conditions, and human errors. For a major bridge, aspects such as value engineering may also play an important role in changes made during the construction phase.

Once the bridge construction is completed, inspection and maintenance play a major role for years to come until a major rehabilitation, replacement, or decommissioning is required. For a bridge to reach its life expectancy of 75 years, inspection and maintenance activities are critical in its life. These are performed periodically or on an as-needed basis. But, until a few decades ago, inspection and maintenance activities used to be an afterthought for many bridge owners. The planning and design stages would normally focus on current needs and initial costs, without much systematic thought given to life-cycle costs related to inspection and maintenance issues. Lately, this has been changing with more emphasis being placed on inspection and maintenance issues.

11.2.1 Historical Background

Inspection and maintenance of structures had been reactive in the past. Maintenance has been mentioned in federal legislation dating back to the 1916 Highway Act passed by the Congress in the United States. All relevant legislation, from the original act passed by the Congress in 1916 that provided for federal aid to highways to the 1970 act, includes the requirements for the maintenance of bridges as well as highways (White et al. 1992). The structural condition and issues identified on the bridge inspection forms are the starting point for successful maintenance activities. But inspection of highway structures, such as bridges, had been relatively a minor part of maintenance work by owners. However, the failure of the Silver Creek Bridge in the state of Ohio in 1967 initiated formalization of inspection procedures. Several other known failures with human casualties, such as the Mianus River Bridge in the state of Connecticut in 1983, Schoharie Creek Bridge in the state of New York in 1987, and the I-35 W Bridge in the state of Minnesota in 2007, caused refinement of these procedures.

In the past, bridge maintenance was a small part of highway maintenance with no dedicated resources. It was partly because of the interstate bridges being relatively new and hence, maintenance was not given serious consideration. This is similar to a young person paying less attention on maintaining their health, given ill effects will not show up 20–30 years later. Bridge maintenance was not done until a serious structural or safety issue surfaced requiring expensive repairs or replacement.

The above facts coupled with increased traffic volumes, after the interstate highway system was built to handle the heavy reliance on automobiles and trucks for goods movement, the heavy loads imposed than anticipated, and the use of deicing chemicals in the North during the winter months have contributed to bridges deteriorating faster and reducing their life span. By the 1970s, the interstate system was showing signs of wear and tear. But because of high inflation and reduced funds in the early 1970s, resources for infrastructure construction were reduced, while the cost of construction and maintenance increased. This resulted in inadequate funding for maintenance. Even though bridge preservation and maintenance have received very high recognition in industry recently, resources are still constrained. At the same time, the data required to quantitatively justify the maintenance needs and effectiveness were also not available. Added to that, most maintenance activities were also not eligible for federal funding until very recently.

11.2.2 Planning and Design

Many decisions taken during planning and design can significantly influence bridge's service life as well as the costs of future inspections and maintenance. Historically, most planning and design operations were governed by current needs with less emphasis on the future. This is mostly because of limitations in the guidelines used that were developed years ago and partly because of lack of data. In most cases, highway bridge designs are governed by specifications such as those developed by the American Association of State Highway and Transportation Officials (AASHTO) that are based on historic demands faced by bridges (AASHTO 2012). Until recently, the specifications were silent on bridge's life span with accepted industry practice being 50 years. Current specifications, based on certain probabilistic assumptions, state bridge's life span as 75 years, but this is mostly based on fatigue considerations with extrapolation of current loading data. Thus, in most cases, designers tend to minimize the cost by optimizing the material use while design details depend on the individual designer's experience and choice.

In general, the standards and the designers were less thoughtful of maintenance aspects that include selection of material and components that are easy to repair, details that can provide easy access to inspect and repair (see Figures 11.6 and 11.7), components that can be replaced easily, and use of features such as integral abutment bridges (see Figure 11.8) to avoid joints that cause deterioration of other components because of their failure, and so forth. In most cases, there is no financial incentive for a designer to spend more time looking for alternatives or providing details that may take more design time, but can improve durability. Most designers probably never are fully aware of bridge maintenance issues because of a very limited interaction between the two groups and a lack of documentation or data on what details are better for bridge maintenance purposes.

Inspection is an important step in determining the structural condition and is the basis for making appropriate decisions on maintenance and rehabilitation operations. Inspection costs include, but are not limited to, personnel costs, access costs, work zone control costs, documentation costs, and evaluation costs. In some cases, the access and work zone control costs can be significantly higher than other costs because of heavy traffic volumes in urban areas. Thus, making access as easy as possible should

FIGURE 11.6 Bridge detail with maintenance access.

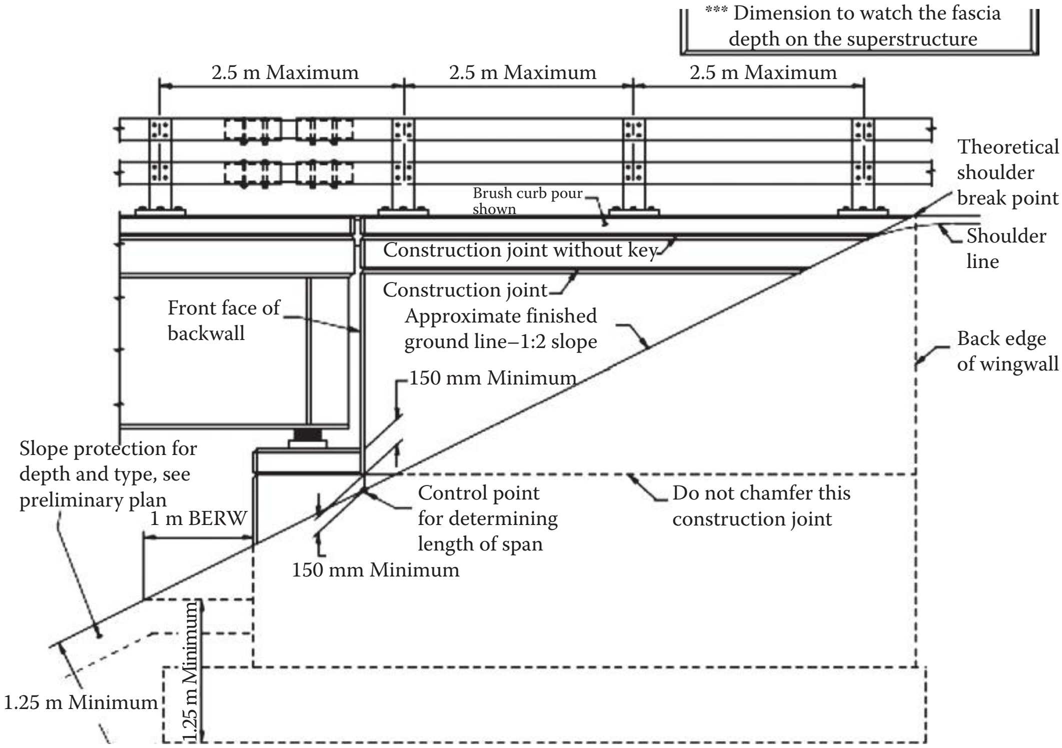


FIGURE 11.7 Bridge detail for inspection and maintenance ease. (Courtesy of the New York State Department of Transportation.)

FIGURE 11.8 Integral abutment bridge. (Courtesy of the New York State Department of Transportation.)

be an important consideration in the bridge design process, but has often been overlooked in the past. In addition, the use of details that make visual observation of critical components difficult should be avoided. The design codes have been relatively silent on these aspects even though *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012) contains some requirements for bridge inspection that the designer needs to consider. The recent AASHTO *Manual for Bridge Evaluation* (AASHTO 2013) documents these requirements in more detail. Both of these manuals contain criteria for determining primary and secondary members and also contain criteria for further determining whether primary members are redundant or nonredundant and whether they are fracture critical or not. Several aspects that can be considered during the design phase to minimize inspection costs and improve inspection quality include (1) bridge type selection, (2) type of details (such as connections), and (3) access (Alampalli and Yannotti 2010).

All the above aspects and lack of emphasis on life-cycle cost approaches have contributed to bridge designs that are less friendly for cost-effective maintenance (or in some cases, no maintenance), thus reducing the useful life span of bridges in the past. But because of the increased emphasis on bridge maintenance and preservation in the last decade, this has been changing slowly. Recent advances in computing technology and materials also had an effect on design and maintenance. Availability of design software and computing power made the design of statically indeterminate structures much easier. This has promoted the use of continuous spans instead of simple spans. This also resulted in designs with greater durability and less maintenance. Given that the state of the art has changed significantly in the last 50 years, bridge engineers know how to design bridges that require less maintenance. Hence, more emphasis is being placed on improving communications between designers and maintenance personnel, and the development of guidelines to assist designers on providing details that improve durability. Review of draft planning and design documents by inspection and maintenance personnel is also becoming popular in some agencies.

Colford (2011) used two elements, half-joints in the longitudinal deck stringer beams and anchorages, on Forth Road suspension bridge in the United Kingdom, to illustrate the value of designing for maintainability. According to the author, the durability, inspection, and maintenance of these two elements were not considered during its design. On the basis of data, the author also concluded that the only effective way that maintenance can truly be considered during the design stage is if the knowledge and experience of the engineers working in the field can be used to improve the future maintainability of bridges.

11.2.3 Construction

Once the design is completed, the projects are normally opened for bidding and a contractor is selected based on lowest bid of construction and fabrication. There is no incentive for a contractor to follow a construction process that maximizes the durability of the structure or minimizes long-term inspection and maintenance costs. In many cases, aesthetics and cost control take precedence over long-term inspection and maintenance issues. Balancing environmental aspects, aesthetics, and security with maintainability comes with a cost, and in such cases maintenance aspects become secondary as the former are controlled by prevailing codes. Maintainability may become one of the primary considerations in the future because of the emphasis on life-cycle costs.

Another aspect is the funding source. Design and construction funding is totally independent of maintenance funding. Thus, there is no incentive for the design or construction personnel to consider maintainability as it does not optimize their funding. Recently, innovative financing methods are increasingly being considered by transportation infrastructure owners. One such method is the design–build–maintain arrangement. In this arrangement, the same entity will be designing, building, and maintaining the bridge for a specified period. Therefore, minimizing the entire life-cycle costs will be important to the profitability of the entity responsible. In such cases, maintainability will have a better chance of being considered as it affects the profitability. At the same time, one should be very careful with this type of funding process. It should be noted that most bridges do not exhibit appreciable loss of service, irrespective of appropriate maintenance is applied or not, for a couple of decades. Hence, maintenance could be completely neglected if the selected maintenance period is relatively short in the design–build–maintain arrangement.

11.2.4 Inspection

Bridge inspection is an essential component of bridge maintenance as the documentation from inspection operations form a basis for maintenance operations, prioritization, funding, and other aspects. The value of bridge inspection in maintenance operations is illustrated by reviewing the original *Manual for Maintenance Inspection of Bridges* developed by AASHTO, working closely with the Federal Highway Administration (FHWA), in 1970. The preface to the second edition of the Manual (AASHTO 1974) notes the following: "This manual has been prepared to serve as a standard and to provide uniformity in the procedures and policies of determining the physical condition and maintenance needs of highway bridges." This manual, along with the training manual entitled *Bridge Inspector's Training Manual* (FHWA 1970), formed the basis for the current bridge inspection program mandated by the Congress through the *National Bridge Inspection Standards* (NBIS). The most recent change to NBIS was made in 2004 (FHWA 2004). The maintenance or other procedures for correcting deficiencies found by the inspectors were not addressed in these manuals or NBIS.

NBIS requires most highway bridges in the United States to be inspected at least once every 24 months to ensure public safety. It is a necessary practice in any structural inspection to identify serious deficiencies affecting public safety, both structural and nonstructural, so that owners can take appropriate action in a timely manner. Thus, NBIS also requires establishment of a statewide procedure to ensure that critical findings are addressed in a timely manner, as needed. It further requires periodically notifying the FHWA of the actions taken to resolve or monitor critical findings. As needed, inspections or monitoring at higher frequencies may be required based on the structural condition observed through these inspections and subsequent evaluations. More information on the current status of bridge inspections was discussed by Alampalli and Jalinoos (2009). The value of bridge inspections in maintenance operations as a part of bridge life cycle, as well as scheduling inspections based on the value gained, can be found in Ettouney and Alampalli (2012a,b).

11.2.5 Maintenance

As discussed earlier, the primary purpose of maintenance is to assure public safety by making sure that bridges are in a condition to provide safe and uninterrupted traffic flow. This includes preventive maintenance and demand (or corrective) maintenance. Preventive maintenance is to ensure structural durability and to avoid unexpected serious deficiencies. This is analogous to taking multivitamins, exercising, and dieting done by human beings to stay healthy, live longer, and prevent sudden, major problems that negatively affect quality of life for long periods. This is discussed in Section 11.2.9.

Because of constrained resources, bridge maintenance has been deferred by several transportation agencies in the last decade. In several cases, federal funding was not allowed for most maintenance operations. This has further accelerated bridge deterioration and a maintenance backlog. Use of new materials, designs, construction methods, and policies has made the maintenance operations more diversified and also extremely important. For example, delamination on concrete decks is more forgiving compared to fiber-reinforced polymer (FRP) decks. If the delamination in an FRP deck is not fixed immediately, the damage can progress rapidly requiring entire deck replacement in one or two winter cycles because of freeze-thaw. Hence, an organizational structure and maintenance reaction mechanism needs further exploring by all the transportation agencies.

The critical findings identified during the inspection of bridges are the preliminary and most important basis for planning maintenance actions. Thus, depending on the organizational structure, bridge management philosophy, and historical developments within each transportation agency, the procedure followed to address critical findings and maintenance approaches varies considerably. For example, in the New York State Department of Transportation (NYSDOT), bridge inspection and maintenance groups are separated. The bridge maintenance engineer is considered the bridge owner. The bridge inspection role is limited to reporting the bridge condition and critical findings (flags) to the bridge maintenance engineer. Inspectors do not make any work recommendations. At the California Department of Transportation (Caltrans), the bridge inspectors do make work recommendations to assist the maintenance crews located in their districts (Barton N., personal communication, 2011). Considering the value of a critical finding procedure for bridge maintenance, the NYSDOT procedure is discussed briefly in Sections 11.2.5.1 through 11.2.5.3.

11.2.5.1 NYSDOT Critical Findings (Flagging) Procedure

The NYSDOT has a systematic procedure in place to identify both structural and nonstructural deficiencies that can affect public safety. The "flagging procedure" has been in place since 1985 and sets forth a uniform method of timely notification to responsible parties of serious bridge deficiencies that require immediate attention as well as issues that if left unattended for an extended period, could become a serious problem in the future. For serious deficiencies, it further establishes requirements for certifying that appropriate corrective or protective measures are taken within a set time frame. The procedure has been updated in 1988, 1994, and 2010 to reflect lessons learned through the years and to accommodate changes in laws and regulations. The entire procedure can be found in the *Bridge Inspection Manual* (NYSDOT 2006) and in the *Engineering Instruction 10-016* (NYSDOT 2010).

11.2.5.2 Flag Types

The critical inspection findings (flags) can be either structural or safety related. The structural flags are further subdivided into two categories: Red Structural Flags and Yellow Structural Flags. Red Structural Flags are used to report the failure of a critical primary structural component or a failure that is likely to occur before the next scheduled inspection. Some examples of these flags include the following:

- Scour that has caused more than minor undermining of an abutment or pier without piles and not found on rock when the danger of failure is imminent or potentially imminent with the next flood (Figure 11.9).
- Distortion in a load path nonredundant member, for example, the visible buckling of a compression chord member in a truss (Figure 11.10).
- Bearings overextended to the point that portions of the superstructure may drop in elevation (Figure 11.11).

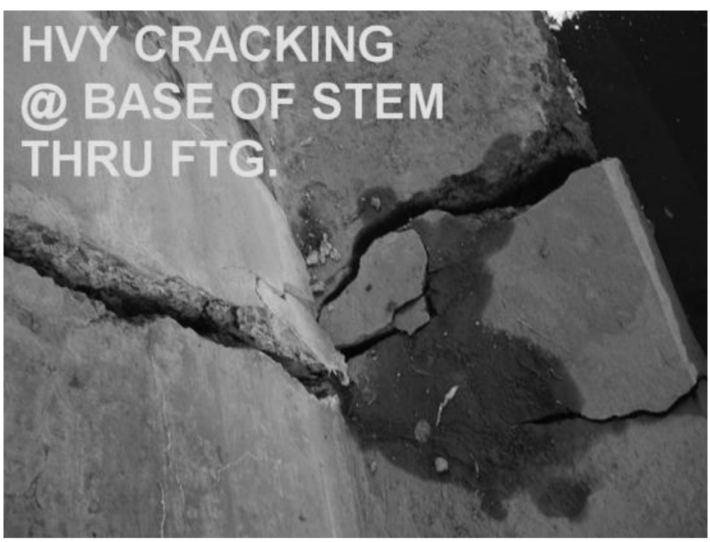


FIGURE 11.9 Red Flag due to serious cracking of abutment stem. (Courtesy of the New York State Department of Transportation.)

FIGURE 11.10 Visible problem in the truss compression member. (Courtesy of the New York State Department of Transportation.)

FIGURE 11.11 Red Flag due to overextended bearing. (Courtesy of the New York State Department of Transportation.)

Yellow Structural Flags are used to report a potentially hazardous condition that if left unattended beyond the next anticipated inspection, would likely become a clear and present danger. This flag is also used to report the actual or imminent failure of a noncritical structural component, where a failure may reduce the reserve capacity or redundancy of the bridge, but would not result in a structural collapse by the time of the next scheduled inspection interval. Some examples of these flags include the following:

- A minor crack in a primary redundant member if it will not propagate in the judgment of the team leader (Figure 11.12).
- A severely cracked and spalled pedestal with some loss of bearing on a multi-girder bridge (Figure 11.13).

FIGURE 11.12 Yellow Flag due to a crack in the primary redundant member. (Courtesy of the New York State Department of Transportation.)

FIGURE 11.13 Yellow Flag due to loss of bearing area in a pedestal. (Courtesy of the New York State Department of Transportation.)

Nonstructural conditions are reported using a "Safety Flag." The Safety Flag is used to report a condition presenting a clear and present danger to vehicle or pedestrian traffic, but is in no danger of structural failure or collapse. Safety Flags can also be issued on closed bridges whose condition presents a threat to vehicular or pedestrian traffic underneath the bridge. Some examples of these flags include the following:

- Concrete falling onto under-feature traffic or onto an area where pedestrians can be present.
- Exposed electrical wiring on light standards on a bridge where pedestrian traffic could be present (Figure 11.14).

FIGURE 11.14 Safety Flag due to exposed wiring. (Courtesy of the New York State Department of Transportation.)

11.2.5.3 Flag Notification and Follow-Up Procedure

The notification and response procedures vary depending on the flag type. More details can be found in the NYSDOT *Bridge Inspection Manual*. Most flags are generated as part of a routine or a special bridge inspection, but some could be generated while reviewing the bridge inspection data or analyzing bridge capacity based on data collected during the bridge inspection. Immediately after the problem is observed, the inspection team leader will complete the required information and generate a report known as a "Flagged Bridge Report" (FBR) which, where practical, includes photographs. If expediency is required, verbal notification to the designated NYSDOT regional bridge engineer is made before completing the FBR. In an extreme case, where an actual failure or clearly perilous condition exists, the team leader has the authority to take immediate measures to close the bridge before notifying the appropriate bridge engineer. The NYSDOT regional bridge engineer then informs the owner of the bridge of the condition on the bridge that requires corrective or protective measures. If the bridge is owned by the NYSDOT, the appropriate regional maintenance engineer is considered the bridge owner for this purpose. Depending on the case, a prompt interim action may be required, where the response determination is made within 24 hours. Depending on the flag type, the owner has a specified time to make appropriate repairs or, based on further analysis, to determine that the condition is safe without any repairs. Once the repairs are made, and before the flag can be removed, certification by a professional engineer is required stating that the repairs adequately address the issue reported in the flag report. If temporary measures are taken to address the flagged condition, while permanent repairs are being scheduled, then the flag is temporarily deactivated. All active structural flags require the structure to be inspected every year or less depending on the condition reported, instead of a 2-year cycle. Generally, all actions for Red Flag conditions should be completed within 6 weeks, but if an action is deferred, a professional engineer shall certify that the bridge is safe and the flagged condition is not a danger to the traveling public.

As a quality assurance effort, the main office bridge inspection unit reviews selected Red Flag summary reports, flag packets, and associated correspondence to evaluate the effectiveness of the inspection program and initiate changes, as needed.

11.2.6 Rehabilitation/Replacement

Beyond preventive maintenance activities, such as bridge washing, the line where maintenance ends and rehabilitation begins is very subjective. In most cases, the degree of repair, funding source, and agency policies dictate if it is called maintenance activity or rehabilitation activity. Maintenance normally implies element level work and fixing/repairing the element/system to improve its condition to operate at a higher level of service without restoring it completely, whereas rehabilitation normally implies increasing the system life through restoring the element to its original capacity. Of course, replacement refers to complete replacement of the system that is beyond maintenance and rehabilitation.

Designers and owners should follow similar principles used in a new bridge design to make sure that rehabilitation does not adversely affect inspection and maintenance capabilities. Wherever possible, the rehabilitation design should increase the maintainability and inspectability of the rehabilitated component as well as the system. This should improve the remaining service life of the bridge and the life-cycle costs.

11.2.7 Role of Bridge Owners and Bridge Characteristics on Maintenance

As noted earlier, all activities from planning to replacement play a major role in safety, serviceability, and life-cycle costs of a bridge structure. Thus, the owner plays a significant role in managing the structure and the network. The main priority of the owner should be to provide the required level of service, while ensuring safety, at a reasonable cost. It is understood that maintaining the lowest life-cycle costs may not be possible at all times because of resource constraints, lack of expertise, and socioeconomic aspects of life. Owners should be aware of the value of bridge maintenance and make sure that all groups that are responsible for bridge management are aware of each other's role through open and constant communication. Before using new materials, construction methods, policies, and so forth, the owner should make sure that the new structure can be inspected and maintained at a reasonable cost by involving maintenance and inspection personnel early in the planning and design stages. A mechanism should be available for maintenance and inspection personnel input in the construction process, especially if any field changes are made after the construction starts. The organizational structure should be reviewed often to make sure that the new and innovative materials/practices can be maintained as required. Appropriate training and technology transfer for maintenance personnel can go a long way in achieving the optimum life-cycle costs.

11.2.8 Effects of Maintenance on Bridge Life Cycle

There have been several studies on the role of maintenance on bridge life cycle. One such study, which has received considerable attention worldwide, is by DeLisle et al. (2004). Based on the NYSDOT historical bridge condition database, they reported that application of corrective maintenance repairs to bridges significantly extends service life. The authors developed deterioration curves that included the effects of both cyclical and corrective maintenance using the following two steps: (1) the age of each bridge was computed from year built to year of last inspection (for bridges that have received no major work) and (2) the condition rating versus age data was then imported into Minitab® statistical software. Regression analyses were performed to provide continuous deterioration equations with 95% confidence intervals. They showed that the corrective maintenance actions can extend service life of steel-girder bridges by as much as 35 years. The figure taken from the NYSDOT *Bridge Management and Inspection Programs Annual Report* (2008a) shows the above in graphical format (Figure 11.15). The report also shows how the bridge treatment costs compare from preventive maintenance to replacement as a bridge deteriorates from new condition to seriously deteriorated condition. The costs range from nominal to $3.7 million.

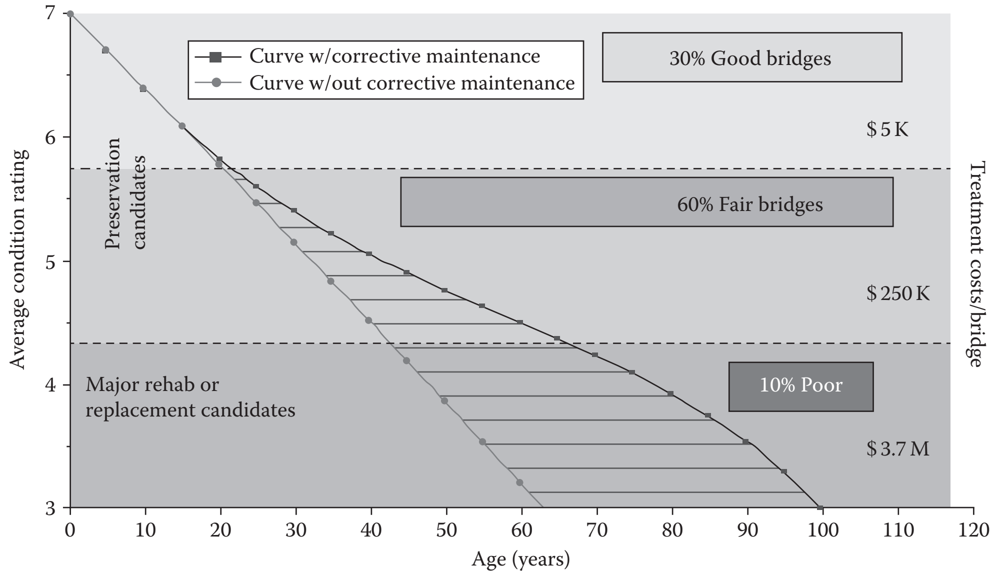


FIGURE 11.15 Effect of corrective maintenance on bridge life span. (Courtesy of the New York State Department of Transportation.)

11.2.9 Maintenance Categories

As noted in earlier sections, bridge maintenance is an important and integral component of effective bridge management. Maintenance activities are normally subdivided into two categories: preventive (cyclic) maintenance and corrective (demand) maintenance. Sections 11.2.9.1 and 11.2.9.2 will further explore these categories in detail.

11.2.9.1 Preventive/Cyclical Maintenance

Preventive maintenance is defined as the recurrent day-to-day, periodic, or scheduled work that is required to preserve or restore a bridge so that it can be effectively used as intended. It includes work to prevent bridge damage/deterioration that otherwise would cost significantly more to restore in the future. The concept of preventive maintenance involves repair of small or potential problems in a timely manner so that they will not develop into expensive bridge replacements (FHWA 2007).

11.2.9.1.1 Types and Description of Preventive Maintenance

Preventive maintenance activities can be subdivided into cyclic or as-needed activities. These activities are performed at regular, predetermined intervals based on the bridge type, the region in which the bridge is located, environmental conditions, the type of the bridge components, the age of the bridge, and so on. Cyclic preventive maintenance includes activities such as the following:

- Bridge washing (Figure 11.16)
- Bridge painting
- Painting structural members
- Bearing lubrication
- Cleaning and resealing expansion joints
- Cleaning expansion bearing assemblies
- Cleaning scuppers
- Sweeping

FIGURE 11.16 Bridge washing. (Courtesy of the New York State Department of Transportation.)

FIGURE 11.17 Crack sealing. (Courtesy of the New York State Department of Transportation.)

- Cleaning grating
- Fixing pavement cracks (Figure 11.17)

As-needed maintenance activities are performed based on observed deterioration during routine or maintenance inspections. These include activities such as the following:

- Concrete deck sealing (Figure 11.18)
- Painting structural members
- Wearing surface repairs (Figure 11.19)

FIGURE 11.18 Concrete deck sealing. (Courtesy of the New York State Department of Transportation.)

FIGURE 11.19 Wearing surface repairs. (Courtesy of the New York State Department of Transportation.)

- Spot painting (Figure 11.20)
- Snow and ice removal
- Sidewalk/curb repairs
- Electrical and mechanical repairs
- Joint repairs

Note that some of these activities can fall into either category depending on performance of the activity and bridge age and condition. For example, painting is purposefully included into both activities as

FIGURE 11.20 Bridge painting. (Courtesy of the New York State Department of Transportation.)

this can be performed cyclically at a given interval or can be done based on observed performance of the paint through the bridge inspection program. The category where it falls depends on the bridge and its regional characteristics as well as the agency's policies and funding availability.

11.2.9.1.2 Preventive Maintenance Recommendations

As noted earlier, the preventive maintenance cycle and how it is applied varies considerably. For example, the NYSDOT-recommended cycle is given in Table 11.1.

Sprinkel (2001) notes that the principal reason for deterioration of concrete bridges is that the reinforcement is damaged by chloride-induced ingression and recommends six maintenance activities to effectively prevent this. The author described these options and their effectiveness in detail in the paper. The strategies include the following:

- Flush debris and chlorides from decks and pier caps annually so chlorides will not be able to penetrate the concrete.
- Repair and seal joints so chlorides will not reach bearings and substructures.
- Seal cracks so chlorides will not reach the reinforcement.
- Patch potholes to keep motorists happy and to prevent accelerated deterioration of the deck.
- Replace the membrane if replacing asphalt overlay.
- Install low-permeability overlays to reduce the infiltration of chlorides. Overlay systems that minimize lane closure time and inconvenience to the public include polymer concrete and very early-strength latex-modified concrete.

TABLE 11.1 NYSDOT Cyclical Maintenance Recommended Activities

Activity	Cycle (Years)
Washing entire structure	1–2
Sealing deck surface and substructure	4–6
Bearing lubrication	4
Painting steel bridge components	12–15
Asphalt wearing surface replacement	12
Seal curb, sidewalks, and fascia	5
Fill cracks and joints	4
Clean drainage system	2

On the basis of the New York City Department of Transportation's (NYCDOT) experience with maintaining more than 750 bridges, about 80% of these bridges having a steel primary structure, Yanev (2003) reported several maintenance tasks and their recommended annual frequency as part of an approach taken by the NYCDOT bridge management unit in modeling the effect of maintenance on bridge condition. Table 11.2 shows the maintenance task and the recommended annual frequency. These were based on empirical evidence and practical considerations predating (Columbia University 1989) and over to the next 10 years (Columbia University 1999).

Irrespective of superstructure and substructure, most bridges have concrete decks and a majority of, if not all, bridge owners spend significant resources in managing and maintaining concrete decks. Carter (1989) notes that preventive maintenance is not only economically feasible but is also required to optimize public investment. He further suggests that in preventing deterioration of concrete bridges, it is as important to seal bridge deck concrete as it is to paint exposed structural steel to prevent corrosion. The author presented the following preventive maintenance procedures used in Alberta, Canada to control the main sources of concrete deck problems (salt and water), and their attempt to extend the lag time between corrosion initiation and spalling.

- Annual bridge washing to remove salt, dirt, sand, and salt-absorbing debris off of gutter lines and curbs.
- Appropriate drainage disposal such that it is carried away from the underside of the deck components.
- Cracks repair: Waterproof shrinkage cracks using saline or siloxane sealers for narrower cracks, and repair wide or actively moving cracks.
- Asphalt sealing: Deterioration can be slowed down by cleaning and drying the surface around all the cracks, and then treating them with a concentrated asphaltic emulsion that fills the cracks, penetrates voids in the asphalt surface, and acts as a binder for a new layer of fine aggregate to restore surface skid resistance.
- Ultrathin polymer wearing surfaces: This is applied to waterproof older decks, especially with wide or flexural cracks. Authors report that permeability tests have shown reduction of permeability by a hundred to a thousand times with this method.

TABLE 11.2 NYCDOT Cyclical Maintenance Activities

Maintenance Task	Cycle
Debris removal	Monthly
Sweeping	Biweekly
Clean drain	Biannually
Clean abutments, piers	Annually
Clean grating	Annually
Clean joints	4 months
Washing deck	Annually
Paint	12 years
Spot paint	3 months
Sidewalk/curb repair	3 months
Pavement/curb sealing	6 months
Electrical maintenance	Monthly
Mechanical maintenance	Monthly
Wearing surface	5 years
Washing underside	Annually

Source: Adapted from Yanev, B., *International Journal of Steel Structures*, 3, 2, 127–35, 2003.

- Concrete sealers: Sealing all exposed concrete bridge surfaces with sealers at a 4-year cycle.
- Use of polymer membranes.
- Use of temporary membranes.

More information on the costs, effectiveness, application methodology, and expected life of each of the presented maintenance activities can be found in Carter (1989) and other similar publications.

11.2.9.1.3 Cost-Effectiveness of Preventive Maintenance

It is not an easy task to obtain information on cost-effectiveness of preventive maintenance activities, mainly because of lack of data management by several transportation agencies. The best information probably is reported by Yanev (2003) based on observed service life of components without any maintenance as shown in Table 11.3. Yanev (2003) assumed that in conjunction with several component replacements, the recommended full maintenance would extend the life of bridges from 30 years to an estimated 120 years. It should be noted that this may not always be possible as bridges are sometimes removed for nonstructural conditions that include functional and socioeconomic reasons.

Even though it is hard to estimate the value of preventive maintenance, it is easily understood that the durability of bridges improve with preventive maintenance if one reviews the average life of components to those presented earlier. Agrawal and Kawaguchi (2009) developed the bridge deterioration curves for the NYSDOT based on the historical condition data of bridges going back to 1991. The data were screened such that major work that could impact the rating was filtered out when the rating of the corresponding bridge element showed a two-point improvement. Using the deterioration curves shown in the report, the number of years it took, for the same elements shown in Table 11.3, to experience a rating change from new condition to seriously deteriorated condition were approximated and shown in Table 11.4 as average life. It may be concluded that the difference between the two columns is an improvement in element life because of maintenance. Note that the data are based on the entire state bridge population and need further work before these findings can be used in bridge management decisions. These findings cannot be applied to individual bridges or projects, but only for network level probabilistic studies.

TABLE 11.3 Shortest Life of Components

Component	Observed Shortest Life (Years)
Bearings	20
Backwalls	35
Abutments	35
Wing walls	50
Bridge seats	20
Primary member	30/35[a]
Secondary member	35
Curbs	15
Sidewalks	15
Wearing surface (separate course)	20/35[a]
Wearing surface (bonded monodeck)	10/15[a]
Piers	30
Joints	10

Source: Adapted from Yanev, B., *International Journal of Steel Structures*, 3, 2, 127–35, 2003.

[a] With/without joints.

TABLE 11.4 Shortest Life of Components

Component	Observed Shortest Life (Years)	Average Life (Years)
Bearings	20	60
Backwalls	35	80
Abutments	35	100
Wing walls	50	80
Bridge seats	20	80
Primary member	30/35[a]	80
Secondary member	35	80
Curbs	15	70
Sidewalks	15	70
Wearing surface (separate course)	20/35[a]	50
Wearing surface (bonded monodeck)	10/15[a]	60
Piers	30	70
Joints	10	60

Source: Observed Shortest Life column is adapted from Yanev, B., *International Journal of Steel Structures*, 3, 2, 127–35, 2003.

[a] With/without joints.

11.2.9.2 Corrective Maintenance

Corrective maintenance is defined as repair and rehabilitation required for improving or extending the life of a component such that the entire bridge (or system) life is not reduced.

11.2.9.2.1 Types and Description of Corrective Maintenance

These activities include, but are not limited to, the following:

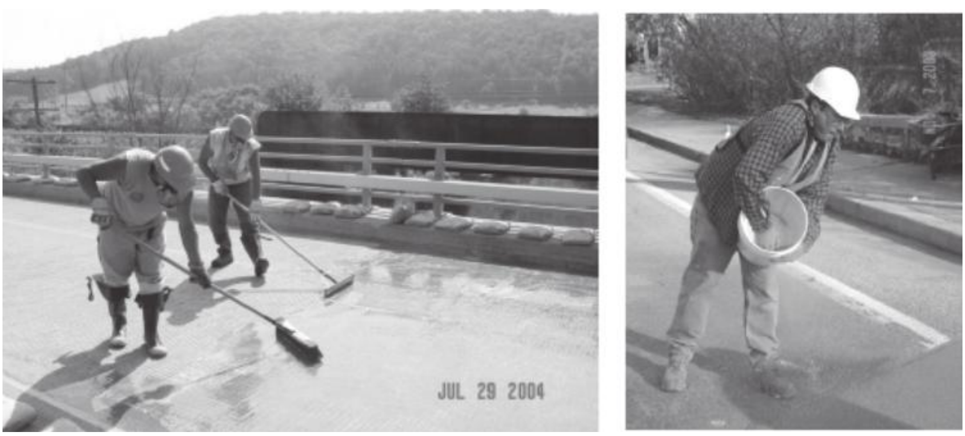
- Repair of delaminated/spalled concrete (piers, columns, beams, abutments, etc.) (Figure 11.21)
- Structural steel or concrete repairs
- Bearing replacements
- Bridge deck wearing surface repairs (Figure 11.22)

In most cases, what differentiates preventive versus corrective maintenance is that corrective maintenance is reactive based on observed conditions during bridge inspection or because of unexpected hazards such as bridge hits and fire. Thus, urgency of corrective maintenance can vary from immediate (in case of a bridge hit or an observed critical finding) that may be highly demanding in some cases (see Figure 11.23 for damage because of a bridge hit) to planned in other cases. Hence, these cases require careful evaluation of the structure as well as the component that needs repair to make sure that the repair is cost-effective. For example, if the estimated remaining service life of the bridge is only 20 years, replacement of a bearing or part of the girder may not be cost-effective and not required if an alternative (such as permanent shoring) that can last for 20 years to match bridge life is available. In most of the cases, these decisions require professional engineering judgment.

11.2.9.2.2 Corrective Maintenance Recommendations

In most cases, corrective maintenance is required for old structures that were designed using materials and details that may not be used on new bridges and design loads based on old specifications that are no longer used. Hence, a good understanding of the structural behavior, bridge history, and experience dealing with in-service bridges is a must for a maintenance engineer responsible for such maintenance actions. In most cases, the structure's current condition is noted in the inspection report without the reason for the cause. A good maintenance engineer should be able to relate the condition with the cause so that an appropriate fix can be made. Then the repair has to be executed using agency personnel or

FIGURE 11.21 Repair of delaminated concrete columns. (Courtesy of the New York State Department of Transportation.)



Spreading the polymer material

Placing aggregate

FIGURE 11.22 Bridge deck wearing surface repairs. (Courtesy of the New York State Department of Transportation.)

FIGURE 11.23 Damage due to vehicle collision. (Courtesy of the New York State Department of Transportation.)

Rating:	3
Description:	Indicates a more serious spalling and delamination problem with about 25 percent of one lane affected and poor riding quality. Wearing surface has cracks or spalls and has a well-worn and polished aggregate.
Cyclical maintenance:	Replace the asphalt wearing surface.
Corrective maintenance:	Repair the concrete wearing surface. Repair the asphalt wearing surface.

FIGURE 11.24 Guidance to maintenance for wearing surface. (Courtesy of the New York State Department of Transportation.)

through contractors without adversely affecting the use of the structure. As noted, in several cases, the maintenance engineer's job is much more difficult than that of the original bridge designer because of the evaluation and field components expertise required to do the job.

Several transportation agencies have prepared guidelines to train and educate their maintenance personnel on appropriate maintenance actions. Figures 11.24 through 11.27 give examples from such a manual prepared by the NYSDOT that provides descriptions and ratings of the structural component that a maintenance engineer may receive from bridge inspection reports along with possible cyclical and corrective maintenance procedures that may be applicable (NYSDOT 2008b). Note that the NYSDOT

Rating:	5
Description:	Only localized areas of leakage (e.g., single longitudinal crack with leakage, or deck edges showing only spotty leakage).
Cyclical maintenance:	Clean the bridge. Seal cracks in the wearing surface. Seal the concrete deck
Deck treatments:	Apply "healer/sealer". Thin polymer overlay. Asphaltic pavement preservation.
Corrective maintenance:	Repair the concrete deck Repair the asphalt wearing surface.

FIGURE 11.25 Guidance to maintenance for structural deck. (Courtesy of the New York State Department of Transportation.)

Rating:	5
Description:	Joint components may show some deterioration such as minor asphalt raveling next to armor angles or filler material deterioration, but the joint is still watertight.
Cyclical maintenance:	Remove debris from the Joint.
Corrective maintenance:	Repair the Joints.

FIGURE 11.26 Guidance to maintenance for deck joints. (Courtesy of the New York State Department of Transportation.)

Rating:	2
Description:	Deterioration of concrete would be significant and characterized by wide-spread mapcracking, efflorescence, spalling and/or scaling. Corroding rebars may be delaminating the rebar cover. There may also be active structural cracks.
Corrective maintenance:	Repair/replace substructure concrete.

FIGURE 11.27 Guidance to maintenance for substructure pier columns. (Courtesy of the New York State Department of Transportation.)

uses a 1 (failed condition) to 7 (new condition) scale for their inspection rating scale and thus the rating shown should not be confused with the 0 to 9 rating scale used by the FHWA.

11.3 New Trends in Bridge Maintenance

Bridge maintenance has evolved significantly in the last decade. More emphasis is being placed on bridge maintenance moving from a very reactive to a proactive, just-in-time approach. As part of these continuing efforts, there has been an increased emphasis on bridge management systems, new technologies, nondestructive testing (NDT) and evaluation methodologies. These topics are covered in Sections 11.3.1 through 11.3.4.

11.3.1 Bridge Maintenance Management

Maintenance management is part of any true bridge management system and is intended to assist bridge maintenance engineers to plan and execute appropriate preservation and corrective maintenance actions at the right time to use the funding in an optimal manner. Ettouney and Alampalli (2012a,b) defined management as a process that follows some sort of decision-making rules combined with some business rules. Thus, the maintenance manager's aim should be the following:

- To develop the right maintenance goals
- To make optimal or effective use of available resources
- To balance short-term needs and long-term durability
- To balance project and network level of service and risks
- To make sound engineering solutions
- To take into account life-cycle considerations

In case of a true management system, the system should incorporate business rules based on agency vision and objectives with life-cycle principles and suggest appropriate actions that are required based on the available resources. These actions could be preservation activities, corrective maintenance activities, or rehabilitation and replacement activities.

For the system to generate an effective suggested program, the following data are required at minimum:

1. Current structural condition: Inspection data play a major role in defining the current structural condition and are a basic tool of bridge management. Thus, the maintenance engineer's role is to inform the inspection group on the type and quality of data, appropriate documentation and level of detail, and required accuracy for various structures and components based on the organizational decision process.
2. Deterioration curves/rate: This is very important in planning the appropriate maintenance activity at the right time and to balance available resources. Historical inspection data with a good record of actions performed on the structure are vital for collecting this information. Idealized and actual inspection rates are shown in Figure 11.28.
3. Available actions and their cost and effectiveness: The maintenance manager should be able to provide possible actions that can be taken based on the structural condition, effectiveness of that action in improving the condition of the component/structure (say increase in condition or life of the repair), cost of the action, resources required, and so on.

Historically, most transportation agencies used their experience coupled with several tools developed internally that account for the agency business rules and condition data to generate preservation activities. But, in recent years, there has been more emphasis to use common tools across the industry to make it easier to maintain and support the tools required given the resource constraints. Pontis (AASHTO 2005a,b) is one such bridge management software that is supported by the AASHTO and is widely used by several states in the United States. It is intended to help a transportation agency or an owner to formulate network-wide preservation and improvement policies for use in evaluating the needs of each structure in a network; and it makes recommendations for what projects to include in an agency's capital plan. It has capability to recommend a preservation strategy that will minimize the cost of maintaining our bridge inventory over the long term.

One of the important features of Pontis (AASHTO 2005a,b) is its ability to develop a network-wide, least-cost investment strategy for maintaining structures in serviceable condition over time based on deterioration rates of elements, given the application of different types of maintenance and repair actions. It has an optimization model that considers the costs of performing different types of repairs on elements in different condition states and determines whether it is more cost-effective to conduct a particular type of repair now or to wait until further deterioration occurs. The result of the model is a set of optimal actions to be taken on each type of structure element in each type of environment in each

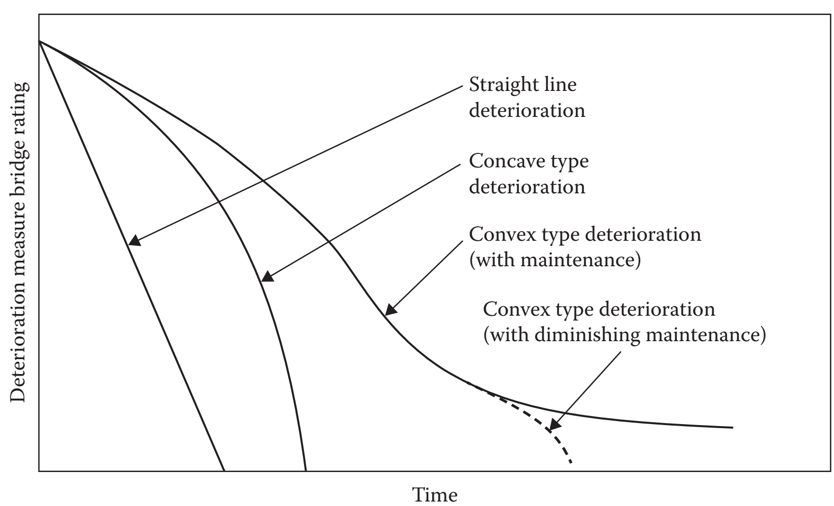


FIGURE 11.28 Generic deterioration curves with and without maintenance.

possible condition state. These model results can then be applied to the structures in the inventory to determine what actions should be taken now (or in each year of a multiyear planning period), along with an estimate of their costs. The model results also include benefits and costs of taking each action, which can be used for setting priorities for use of limited resources.

In essence, if all the business rules of the agency, available actions and their costs, and structural conditions are incorporated properly, the management system will be able to give possible actions the agency should take for each structure to maintain the performance measures defined by the agency. It should be noted that the outcome of the system is directly dependent on the type and accuracy of the input data. A maintenance engineer should use the outcome as guidance and make appropriate changes based on his/her experience, common sense, and socioeconomic issues to meet the agency goals.

Recognizing the value of maintenance to bridge management efforts, the following deserve attention:

- Definition or categories of maintenance
- Components requiring maintenance
- Type and scope of maintenance efforts
- Frequency of various maintenance efforts
- Optimization of maintenance efforts and the associated cost, without affecting the required performance levels and durability

Given the amount of money spent on maintenance, the maintenance management systems are still in their infancy with a lack of serious effort by most transportation agencies and owners in documenting their efforts. More work is needed in this area.

11.3.2 Role of New Technologies

Innovations in materials used for bridge components, construction methodologies, financial schemes, management methods, and repair technologies have been evolving rapidly in recent years. At the same time, because of the mobility and reliability expected from the transportation system, there has been tremendous pressure on owners to minimize the traffic interruptions and highway/bridge closures to a minimum during their life spans. This has a tremendous impact on the way bridges are built and the way bridge inspection and maintenance activities are performed. These innovations have extended to maintenance activities. At the same time, there are several constraints on maintenance engineers that include limited personnel and financial resources, time constraints because of increasing restrictions (when and for how long) on bridge closures, and more resources consumed by demand repairs than preventive maintenance because of advanced deterioration faced by several bridges.

NDT and structural health monitoring (SHM) has been advocated more and more recently to assist the maintenance engineers in conducting their job more efficiently and this is covered briefly in Sections 11.3.3 and 11.3.4. Some aspects that need to be considered in this environment are the following:

- Organizational structure
- Designing for maintainability
- More emphasis on training and technology transfer
- Data collection and sharing on performance of various materials and technologies

11.3.3 Role of Nondestructive Testing and Evaluation

The NDT has been on the forefront for bridge inspection and maintenance management in recent years because of the quantitative information required to optimize the available resources. The NDT methods have been increasingly used by bridge maintenance engineers to augment data collected by bridge inspectors that are primarily visual. Although visual inspections can provide an overall picture of the component under consideration, it is very hard to use that information in deciding the extent and scope of maintenance activity. This is especially true for components such as bridge decks where one rating is given for the entire deck. To develop an appropriate repair strategy, bridge maintenance engineers need more information to decide the condition of the deck at various stages. Quantitative data can also be used to prioritize maintenance activities within the network when several of the bridges (or components) have the same rating based on visual inspections when available personnel and financial resources are limited.

The NDT methods also have been used to decide if a cyclical maintenance activity is required or not. For example, a sealer is applied routinely every 4–6 years, but if an effective NDT method can indicate that the sealer is still effective, then the frequency of the application can be reduced. These methods can also be used for quality control and quality assurance of maintenance repairs.

Some of the NDT methods that have been considered by maintenance engineers recently include the following:

- Sounding
- Half-cell potential surveys
- Infrared thermography: For bridge deck condition surveys, loose concrete on the underside of the deck
- Ground-penetrating radar
- Impact echo
- Sounding
- Ultrasonics

These methods are discussed in detail with examples in the literatures including Ettouney and Alampalli (2012a,b) and Jalinoos et al. (2009). Before the application of any NDT method, one has to evaluate the benefits and costs to make sure that appropriate value is gained using these methods. Such methods can be found in the literature (Ettouney and Alampalli 2012a,b).

11.3.4 Role of Structural Health Monitoring

The main challenge of a maintenance engineer is to execute appropriate maintenance activities with minimum expenditure. This involves just-in-time delivery of the activity with the right scope based on the need. Considering bridge maintenance has significant value in effective bridge management, as noted earlier in the chapter, the following deserve attention:

- Category of maintenance
- Components requiring maintenance
- Type and scope of maintenance
- Frequency
- Optimization of maintenance efforts and associated cost

With advances in information technology, sensors, and instrumentation for monitoring, the SHM can be used effectively to address some of the earlier issues.

11.3.4.1 Cost–Benefit (Value) Methodology

The SHM, if used properly, has a high potential for scheduling maintenance activities just-in-time and scope. The main reason hindering the wide use of the SHM for bridge management has been the lack of cost–benefit data and a quantitative approach. Realizing this gap, Alampalli et al. (2005) advocated a cost–benefit (value) approach to evaluate the usefulness of the SHM before its implementation for bridge management applications.

This method can be greatly simplified by the following steps:

- Estimate the cost of activity(s) considered for possible assistance from the SHM.
- Estimate the cost of the same activity(s) using the SHM.
- Estimate the cost savings or the value added.

- Estimate the break-even period.
- Advocate the use of the SHM, if the break-even period is acceptable to the owner.

Quantification of costs and benefits are needed in the earlier evaluation. These can be based on data available in agencies, experience of the maintenance engineer, or data available in the literature. Costs are relatively easy to obtain whereas quantifying benefits is more difficult. One way to define maintenance value is to quantify the cost of not performing maintenance activities. More details on how to use this methodology can be found in Ettouney and Alampalli (2012b).

11.3.4.2 Illustrative Example

Use of SHM for a typical maintenance application is briefly described in this section to illustrate the earlier methodology (Alampalli et al. 2005). A closed-circuit television (CCTV) can be used on a strategic location of a bridge to schedule activities such as debris removal, washing, and cleaning. By monitoring the CCTV at regular intervals, from a remote site, the maintenance tasks can be performed at required intervals, as needed.

Let us assume such a scheme was considered for a midsize bridge to observe maintenance needs such as removing debris, cleaning drains, and cleaning expansion joints and gratings. Let us further assume the following:

- Cost of cleaning expansion joint and grating is $2,000 and is generally performed every month (or 12 times a year).
- A CCTV costing $10,000, designed to monitor this activity, was considered for installation to optimize the above activity.
- Because of the SHM, it is determined that the same activity can be performed, on average, once in 1.5 months (eight times a year).

Then, the cost savings because of the SHM will be about $8,000 per year, with break-even point of 15 months (cost of CCTV/savings per year = $10,000/$8,000 = 1.25 years). This is illustrated in Figure 11.29. After 3 years, the SHM-assisted maintenance would save almost $14,000. If the owner is satisfied with the break-even period of 1.25 years, then the SHM should be used for the earlier activity. Potential cost savings of the SHM-assisted activities will vary depending on the cost of the SHM equipment and maintenance, number of activities that can be covered, and personnel costs. Note that for

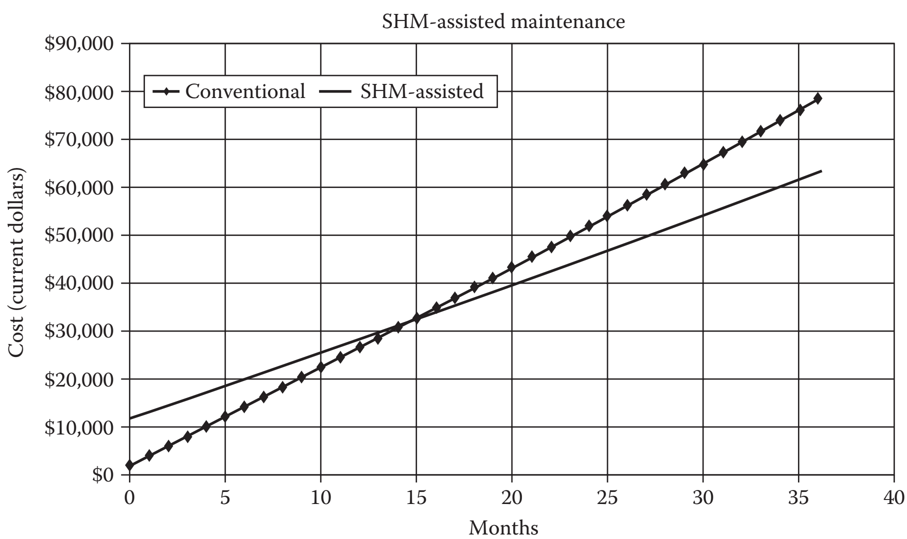


FIGURE 11.29 Break-even analysis for use of SHM for bridge maintenance.

simplicity, cost of operating the SHM, discount rate, and so on, were not considered in this example. All these should be properly accounted for to make informed decisions.

The use of the SHM does not always provide cost savings. It may increase the attention the structure requires and the maintenance period of an activity may have to be shortened (i.e., performed more often) based on the data obtained from the SHM. In those cases, the value of increased safety or reliability should be properly accounted for in making a decision on using the SHM for that application.

The earlier example illustrates the process in a simplified manner. This process (Figure 11.30) can be extended to any maintenance or management activity in a simplified or at a detailed level depending on the choice and purpose of the decision-maker.

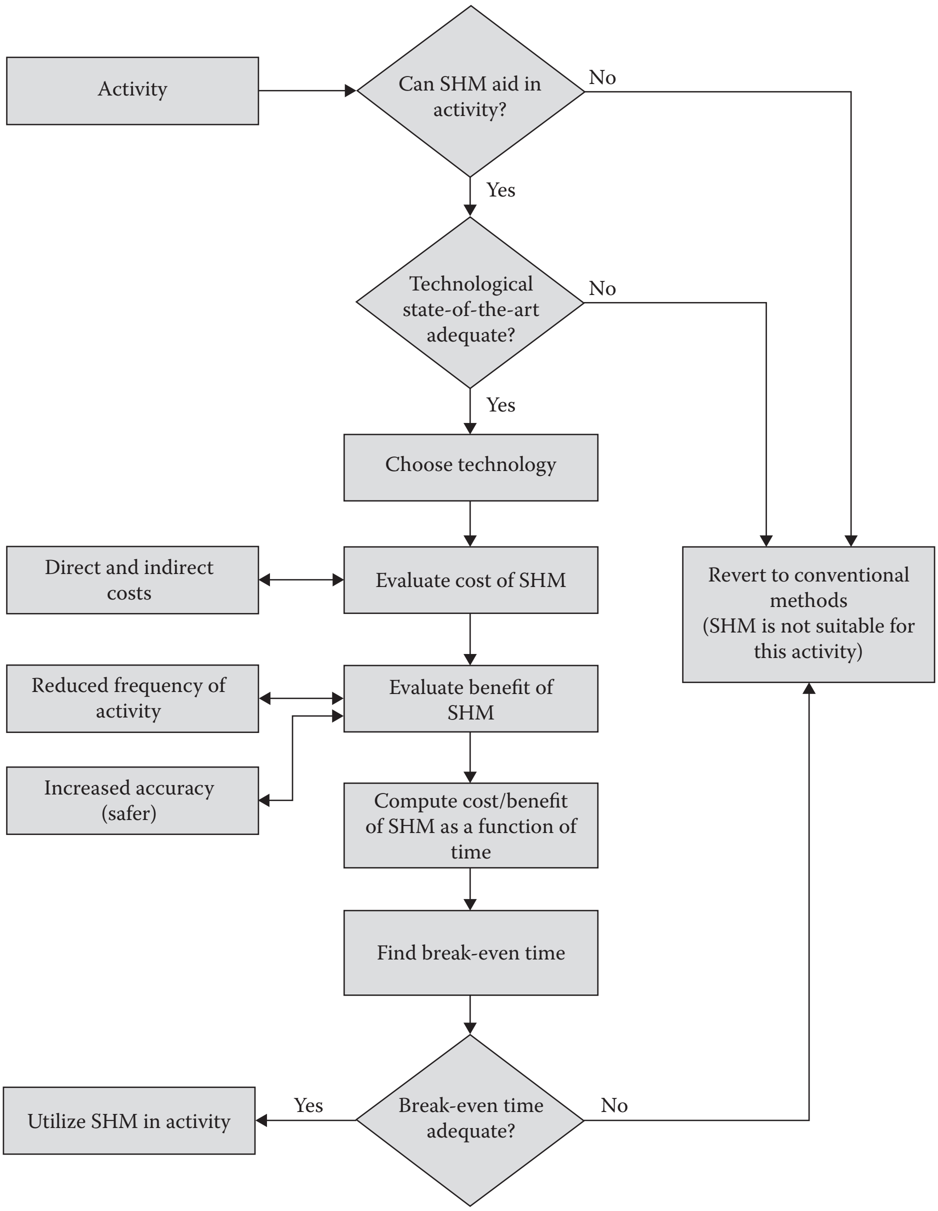


FIGURE 11.30 Evaluation process for SHM use for bridge management.

11.4 Summary

This chapter reviews the history of bridge maintenance, current status, and its future. Bridge maintenance has a major role to play in the future of effective bridge management because of the continuously deteriorating bridge infrastructure. Better communication between the maintenance engineers and other stakeholders (such as planners, designers, construction engineers, and inspectors) is required to make sure that bridges can be properly and cost-effectively maintained. This is essential given the resource constraints faced by bridge owners. These efforts coupled with development of good maintenance databases to support bridge management systems can lead to bridges with improved durability and resiliency while minimizing life-cycle costs. The new materials and technologies, the SHM, bridge management systems, and the NDT methods have a potential to play a major role in shaping the future of the bridge maintenance in the upcoming decades.

Acknowledgments

The opinions expressed in this chapter are those of the author and not necessarily of the organization the author represents. The author acknowledges Linda Banks of the New York State Department of Transportation and Sharada Alampalli of Prospect Solutions LLC, for their review of this chapter and suggestions they made to improve the chapter. The author also acknowledges Sandeep Alampalli for his patience and support during this chapter preparation.

References

AASHTO. 1970. *Manual for Maintenance Inspection of Bridges*. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 1974. *Manual for Maintenance Inspection of Bridges*. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2005a. *Pontis Release 4.4 User's Manual*. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2005b. *Pontis Release 4.4 Technical Manual*. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2012. *AASHTO LRFD Bridge Design Specifications*, Customary U.S. Unit, 2012. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2013. *The Manual for Bridge Evaluation*, 2nd Edition, with 2011 and 2013 Interim Revisions. American Association of State Highway and Transportation Officials, Washington, DC.

Agrawal, A.K., and Kawaguchi, A. 2009. *Bridge Element Deterioration Rates: Final report*. New York State Department of Transportation, Albany, NY.

Alampalli, S., Ettouney, M. M., and Agrawal, A. K. 2005. Structural health monitoring for bridge maintenance. *Bridge Structures: Assessment, Design and Construction*, 1(3), 345–54.

Alampalli, S., and Jalinoos, F. 2009. Use of NDT technologies in U.S. bridge inspection practice. *Materials Evaluation,* American Society for Nondestructive Testing, 67(11), 1236–46.

Alampalli, S., and Yannotti, A. P. 2010. *Designing Bridges for Inspectability*. In Structural Materials Technology (SMT): NDE/NDT for Highways and Bridges Topical Conference, August 16–20, American Society for Nondestructive Testing, New York, NY.

Carter, P. D. 1989. Preventive maintenance of concrete bridge decks. *Concrete International*, 11(11), 33–6.

Colford, B. R. 2011. *Bridge Design for Maintenance*. In New York City Bridge Conference, July 25–26, New York, NY.

Columbia University. 1989. *Preventive Maintenance Management System for New York City Bridges*. Technical Report, Columbia University, Center for Infrastructure Studies, New York, NY.

Columbia University. 1999. *Preventive Maintenance Management System for New York City Bridges*. Technical Report, Columbia University, Center for Infrastructure Studies, New York, NY.

DeLisle, R., Shufon, J., and Adams, L. 2004. *Development of Network-Level Bridge Deterioration Curves for Use in NYSDOT's Asset Management Process*, Paper No. 04-3397. Presented at the Transportation Research Board annual meeting, January 11–15, Washington, DC.

Ettouney, M. and Alampalli, S. 2012a. *Infrastructure Health in Civil Engineering: Theory and Components*. Volume I. CRC Press, Boca Raton, FL.

Ettouney, M. and Alampalli, S. 2012b. *Infrastructure Health in Civil Engineering: Applications and Management*. Volume II. CRC Press, Boca Raton, FL.

FHWA. 1970. *Bridge Inspector's Training Manual 70*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

FHWA. 2004. National Bridge Inspection Standards, 23 CFR Part 650. Federal Highway Administration, U.S. Department of Transportation, Federal Register, Vol. 69, No. 239.

FHWA. 2007. *Bridge Maintenance Training Reference Manual*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Jalinoos, F., Arndt, R., Huston, D., and Cui, J. 2009. Structural Health Monitoring by Periodic NDT: NDT for Bridge Maintenance, *Materials Evaluation*, 67(11), 1300–1307.

NYSDOT. 2006. *Bridge Inspection Manual* (1997 with updates through 2006). New York State Department of Transportation, Albany, NY.

NYSDOT. 2008a. *SFY 2007-08 Annual Report of Bridge Management and Inspection Programs*. New York State Department of Transportation, Albany, NY.

NYSDOT. 2008b. *Fundamentals of Bridge Maintenance and Inspection*. New York State Department of Transportation, Albany, NY.

NYSDOT. 2010. *Inspection Flagging Procedure for Bridges*. In Engineering Instruction 10-016, New York State Department of Transportation, Albany, NY.

Sprinkel, M.M. 2001. Maintenance of concrete bridges. *Transportation Research Record: Transportation Research Board*, 1749, 60–3.

White, K. R., Minor, J., and Derucher, K. N. 1992. *Bridge Maintenance Inspection and Evaluation*, 2nd edition. Marcel Dekker, New York, NY.

Yanev, B. 2003. Management for the bridges of New York City. *International Journal of Steel Structures*, 3(2), 127–35.

12

Nondestructive Evaluation Methods for Bridge Elements

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12.1 Background

12.1.1 National Bridge Inspection Standards

Following the collapse of the Silver Bridge in 1967, the U. S. Congress passed the Federal Highway Act of 1968, which led to the creation and implementation of the National Bridge Inspection Standards (NBIS). These standards define the Federal requirements for publically owned highway bridge inspections in the Code of Federal Regulations, Title 23, Part 650. These standards establish requirements for inspection procedures, inspection frequency, personnel qualifications, inspection reports, and inventories. The standards also delineate the five different types of inspections: routine, in-depth, interim, damage, and inventory. These different types of inspection are implemented based on the purpose of the inspection and the condition (age, traffic conditions, and known deficiencies) of the bridge. The maximum interval for inspections is typically 24 months; for certain bridges with advanced deterioration or known deficiencies, shorter inspection intervals may be used, at the discretion of the bridge owner. The inspection interval may increase to 48 months for certain bridges that are in good condition and have adequate load-carrying capacity, relatively short span lengths (100 ft. or less), and redundant load paths (FHWA 1988). The first full biennial inspection cycle began in 1973, and this cycle continues to the present day.

The most recent revisions made to the NBIS were completed in 2004 and implemented in 2005 (FHWA 2004; Alampalli and Jalinoos 2009). The revisions clarify that the responsibility for ensuring that inspections are carried out within a state belongs to that state's Department of Transportation (DOT), and inspection frequencies are described in terms of months. These revisions include requirements for systematic quality control (QC) and quality assurance (QA) to ensure consistency in inspection results and periodic refresher training for inspectors. The revision also required statewide procedures to ensure critical findings are addressed in a timely manner.

There are variations in the methods of implementation of the NBIS requirements between individual states, with different states using different approaches to data collection and recording during inspections, different qualification requirements for team leaders, different access requirement during inspection, and so on. As result, there is variation in the outcome of inspections between the different states, simply because they conduct the inspection differently. In addition, subjective rating scales used to record the inspection results according to NBIS requirements can have different interpretations between different states, and even between different inspectors in the same state. A study conducted by the Federal Highway Administration (FHWA) in 2001 indicated that there were inconsistencies in inspection results among a group of inspectors from different states across the United States (Moore et al. 2001).

The results of a bridge inspection are generally recorded using a condition rating for a bridge component or element and supporting notes and photographs illustrating conditions found during the inspection. The condition ratings usually stem from visual observations of the condition of the bridges, sometimes supplemented with hammer sounding of concrete. The use of nondestructive evaluation (NDE) or nondestructive testing (NDT) is limited during routine inspections. NDE technologies are usually implemented to address specific forms of deterioration or damage during in-depth inspections or special inspections that are conducted as a result of visual findings from routine inspections. For example, if cracks were found visually in a steel bridge, NDT may be conducted to better define the extent of cracking in the bridge. Concrete bridge decks that are deteriorating may have NDT performed to prioritize repair activities or better define future programmatic needs. However, NDE/NDT technologies can play a role in improving the reliability of visual inspection in general, by providing more quantitative data on the damage present or by improving the effectiveness and reliability of the inspection. Previous research has shown that visual inspections have certain limitations, and NDE/NDT can improve the reliability of, for example, crack detection in steel bridges or condition assessment of bridge decks, as well as condition assessment of other bridge elements. The technologies can also be used to detect damage in its embryonic stages, before it manifests in visually observable effects, such that repair and rehabilitation can be completed before the damage affects the serviceability or safety of a bridge. In some cases, NDE/NDT technologies can detect critical damage that cannot be detected using visual inspection. In these cases, NDE/NDT may provide the only means of ensuring the safety and reliability of a bridge.

Because of the important role that NDE and NDT technologies play in ensuring the safety and serviceability of bridges, this chapter describes key technologies that are available for assessing bridge conditions. These technologies are generally intended to assess common forms of damage in bridges, such as cracks in steel bridge elements or delaminations in concrete bridge elements.

12.1.1.1 Rating Approaches for Visual Inspection

There are several different methodologies currently being utilized for describing the condition of bridge and bridge components/elements. Generally, there are two common rating methodologies used for recording the condition of highway bridges: component-level rating that records the overall condition of the primary components of a bridge and element-level rating schemes that rate individual elements of the bridge. The component-level rating scheme is described in the FHWA Recording and Coding Guide (FHWA 1995). The element-level scheme is described in the Commonly-Recognized Bridge Elements (CoRE) guide and/or the Bridge Element Inspection Manual, developed by the American Association of State and Highway Transportation Officials (AASHTO 1997; Jensen and Johnson 2010). Although each state is required to submit inspection findings to the FHWA according to the Recording and Coding Guide, as indicated in the NBIS, the use of element level schemes is increasingly common to meet the needs of states in terms of managing their bridge inventories. Among the element-level approach, there are variations between states implementing the methodology.

Under the NBIS (component-level) system, each major component of the bridge (superstructure, substructure, and deck) receives a single rating to describe its overall condition. The ratings are applied on a scale from 0 to 9, with 9 representing a component in "Excellent" condition, 6 for a component in "Satisfactory" condition with minor damage, and 3, "Serious" condition for a bridge with significant deterioration or damage, and 0 representing "Failed" condition (FHWA 1995). The rating scale is intended to describe the general condition of the components of the bridge, in terms of the severity of damage or deterioration to that component and its effect on the overall condition of the component.

The AASHTO CoRE guide specifies typical elements common to highway bridges for use in element rating systems (AASHTO 1997; Jensen and Johnson 2010). Originally published in 1994, a new CoRE guide was approved in 2010 for use in element-level inspection schemes and associated bridge management programs. The guides are applied in element-level inspection schemes in which bridge elements—for example, the steel girders—are individually rated using *condition states*. Using such a scheme for inspection documents the type of damage and the extent to which that damage exists on the individual elements of a bridge. For example, when rating steel girders, the inspector is required to provide estimates (in linear feet for a girder) of damage—for example, corrosion—under an element-level inspection scheme. The conditions of elements are described in four general states composed of good, fair, poor, and serious conditions. Defect or smart flags are used to break down the deteriorated quantities into specific defects that are driving the deterioration of an element or to identify forms of damage, such as fatigue cracks or severe section loss, which may require urgent action.

Although component-level and element-level rating schemes are commonly used, some states use variations on these approaches to rate bridges in their inventory. For example, New York State rates its bridges and bridge elements on a 7-point scale, with 1 representing a totally failed condition, 3 representing serious deterioration or not functioning as originally designed, 5 representing minor deterioration but functioning as originally designed, and 7 representing an element in new condition with no deterioration. The even-numbered ratings—2, 4, and 6—are used to shade between the odd-numbered ratings. Ratings for various elements within each span of the bridge are determined and then an overall rating is given to each span (NYSDOT 1997). As such, the systems utilize portions of the NBIS rating methodology, in terms of describing the effects of damage or deterioration on the safety condition of the bridge elements and portions of the element-level approach, because these ratings are applied on an element span-by-span level as opposed to the single overall rating for the component provided by the NBIS. Similar to the smart flags used in the element-level approach to identify forms of damage that may be critical, the system uses colored flags describing the urgency of actions for certain serious conditions.

On a national level, the direction of condition rating schemes is heading toward element-level inspections utilizing as few as four condition states, as described in the AASHTO Bridge Element Inspection Manual. In the future, the NBIS may be converted to an element-level inspection scheme, and efforts to support this change are currently underway at the FHWA (Everett 2011). This change in the NBIS rating methodology is motivated largely by the need for more comprehensive information on bridge condition to support planning and oversight on the national level. The limitations in the reliability of the component rating system, as described in Section 12.1.2, also contribute to the need for updating the NBIS rating approach. NDE technologies can play an important role in improving the reliability of the ratings be enhancing and quantifying bridge conditions and supplementing visual inspection results.

12.1.2 Reliability of Visual Inspection for Highway Bridges

This section provides an overview of the 2001 FHWA report, "Reliability of Visual Inspection for Highway Bridges" (Moore et al. 2001). This study was the first systematic study on the subject of visual inspections in highway bridge inspection since the inception of the NBIS. Its stated objectives were to assess the overall reliability of both routine and in-depth visual inspections, to investigate the key factors that affected the reliability of these inspections, and to explore the ways in which visual inspection procedures differ between individual states' DOTs. (Moore et al. 2001). The study focused on the reliability of visual inspection as a test method for condition assessment of bridges, as implemented across the federal-aid program.

Forty-nine inspectors from 25 states participated in the study, typically working independently to conduct inspections on the test bridges. In general, the study consisted of inspectors being tasked to complete an inspection meeting the NBIS requirements. Inspection forms were provided for most of the tasks, such that the inspectors completed the inspection and documented results on standard forms. Inspectors were not provided with historical data, such as prior inspection reports or detailed inspection procedures or prioritization information. They were provided with general information about each structure and instructions for completing each inspection task.

One conclusion of the study was that there was significant variation in the time that inspectors spent doing a particular task. This conclusion indicated that there was significant variation in the inspection procedures and practices among the state's participating in the study. It was also found that there was significant variation in the way routine inspections were conducted, resulting in a variation in the condition rating assigned and the documentation of inspection results. For example, out of nine available condition ratings, an average of between four and five different condition ratings values were assigned to each bridge component, with a maximum of six.

Statistical analysis of the routine inspection results indicated that only 68% of inspection results varied within ±1 of the average rating assigned (Moore et al. 2001). A sample of these results is shown in Figure 12.1. As shown in the figure, which documents the NBIS component ratings for the deck, superstructure, and substructure for the single-span concrete T-beam bridge, there was significant variation in the component ratings for this simple, common bridge. Rating results ranged between 2 and 7 for the superstructure of the bridge, in common language, the rating results range from "critical" (2) to "good" (7) condition.

The study also found that in-depth visual inspections were unlikely to identify the defects that the inspections are specifically intended to identify. This includes a number of defects that could be expected to be observed during a thorough in-depth inspection, such as corrosion, missing rivet heads, coating failure, and importantly, cracks. For one task of the study, which involved an hands-on inspection of one bay of a welded plate girder bridge, there were seven areas where there was a crack indication in a weld. Based on the testing results, a detection rate of only 4% was found, with many inspectors not reporting any crack indications in the area inspected. The thoroughness of the inspections conducted, distance to target (distance at which the surface is observed by the inspector), and the use of a flashlight during by

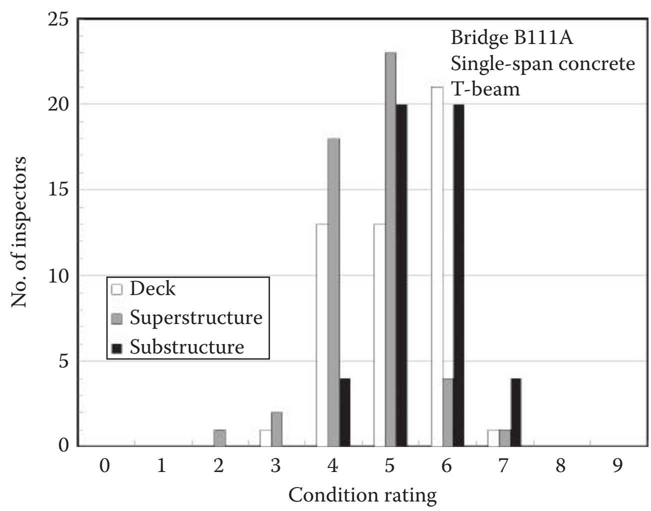


FIGURE 12.1 Condition rating distribution for a concrete T-beam bridge. (From Moore, M., B. Phares et al., *Reliability of Visual Inspection for Highway Bridges*, Federal Highway Administration, McLean, VA, 2001.)

the inspector were factors thought to contribute to the low crack detection rates. NDE technologies such as those described herein may provide more reliable crack detection than was found during the study.

In-depth sounding of a bridge deck was also completed as part of the study, and these results indicated that there could be significant variation in sounding results from different inspectors assessing the same bridge deck. During this testing, a deck of a bridge that was closed to traffic was assessed by inspection teams using chain-dragging and hammer sounding. NDE technologies such as ground-penetrating radar (GPR), impact echo (IE), and infrared (IR) thermography are intended to improve the ability to assess the condition of bridge decks and other concrete components and could improve the consistency and accuracy of such deck evaluations. Perhaps more importantly, for the case of GPR and IR, assessment can be completed with limited or no traffic control, as opposed to sounding or IE, which require traffic disruptions to access the decks for assessments.

It was concluded that the NBIS allowed for some significant variation in the results of visual inspections of bridges between states and that there was both room for and need for improvements in the consistency of visual inspection procedures for bridges. The outcome of this study resulted in modifications to the NBIS—specifically, increased requirements for QA and QC to ensure consistent results across individual states' inspection programs and required retraining of inspectors. These modifications were intended to reduce the variation in inspection results across a given bridge inventory and increase the training to inspectors with the goal of obtaining more thorough, effective, and consistent inspection practices.

This research illustrates the need for NDE/NDT technologies as a part of an effective and reliable bridge inspection program. Key goals of utilizing an NDE technology generally include the following:

- Increased reliability of damage detection
- Reduced traffic control requirements
- Reduced cost of inspections
- Reduced time and access requirements
- Better quantification of damage extent and criticality

The following sections describe some of the key NDE technologies available for bridge condition assessment. The chapter is divided into two major sections: NDE methods for steel elements and NDE methods for concrete elements.

12.2 Nondestructive Evaluation Technologies

There are many NDE/NDT technologies in existence, many of them experimental or highly specialized, while others have long-established histories, standardized procedures, and established inspector certification processes. Widespread implementation of NDE/NDT technologies as part of routine inspections is limited, although its use is increasing as the bridge inventory ages and maintenance of the existing infrastructure becomes the focus of transportation programs. This chapter reviews NDE/NDT technologies for the condition assessment of concrete and steel bridge elements, focusing on existing technologies that can be applied to bridges. Most of these technologies are intended to detect and characterize specific damage modes that are typical for highway bridges, such as steel section loss due to corrosion, cracking due to fatigue, or corrosion damage of concrete. These technologies have the ability to be used in the field during bridge inspections to improve the capability of the inspector and the reliability of the inspection results. First, existing technologies for NDE/NDT or steel bridge elements are discussed, followed by methods applicable to concrete bridge elements. The chapter concludes with a description of the future directions for bridge inspection programs and inspection planning.

12.3 Nondestructive Evaluation for Steel Bridge Elements

This section discusses NDE technologies that can be used for the condition assessment of steel bridge elements. The primary damage modes affecting steel bridges are corrosion damage (section loss) and cracking. There are a number of standard and well-developed technologies for crack detection in steel structures, including liquid dye penetrant inspection (PT), ultrasonic testing (UT), eddy current testing (ET), magnetic particle testing (MT), and radiographic testing (RT). Of these methods, RT, MT, and UT are standard methods for the fabrication of steel bridges and are commonly used in fabrication shops for QC of welding (AWS 2010). These technologies also have field applications, with PT, MT, and UT being the most widely used methods for in-service inspections; PT and MT for crack detection and UT for crack detection and steel thickness measurements (Moore et al. 2001; Alampalli and Jalinoos 2009). The use of RT in the field is limited, due to limitations for detecting tight cracks, safety issues associated with the technology, and access requirements to implement the technology.

The ET method has also been used for bridge inspection in the field historically (Lozev et al. 1997; Washer et al. 2000). This method uses a small sensor to induce electromagnetic (EM) fields in the surface of the steel and detects changes in that field resulting from a crack. While a well-established method, use of this technology in the field has been more limited than PT, MT, and UT. The primary advantage of ET over these other technologies is its ability to detect cracks through coatings, allowing inspections to be performed with minimal surface preparation and without destroying intact coatings systems.

Experimental technologies for crack detection and monitoring of cracks in steel bridges include coating tolerant thermography, which utilizes transient heat flow to reveal cracks, and advanced eddy current sensors that use meandering winding magnetometers (MWM) to detect and monitor crack growth (Zilberstein et al. 2003; Lesniak et al. 1997). An electrochemical fatigue sensor has also been developed, which is intended to monitor if a fatigue crack is growing (Miceli et al. 2010). Vacuum-based sensor technologies have also been developed and tested for aerospace structures (Chiclott and Roach 2010). Among these innovative new technologies, the MWM likely has the most field use in other industries, being used for pipe and aircraft inspection. Field testing on bridges has been limited. However, although new technologies are being developed for crack detection, when access to the surface of steel being inspected is possible, visual inspection and existing NDT methods such and MT, PT, and ET provide suitable and well-established methods for crack detection. When access to the area being inspected is limited, UT is a common method used for crack detection.

Another experimental technology is ultrasonic-guided waves, which can be used to detect deterioration and defects in inaccessible areas of plates, such as interior plates in a multiplate connection. Guided by the plate thickness, these waves can propagate over long distances. In contrast with traditional UT, which uses high-frequency pulses and small wavelengths, guided waves use long wavelengths (lower frequencies) that are resonant in the plate of a particular thickness (Rose 2002; Alleyne et al. 2004; Matzkanin and Yoken 2009). Typical applications of guided wave ultrasonics include the detection of section loss in pipes that are inaccessible, either due to access restrictions to an area or due to insulation covering the pipe. Guided waves have also been tested to detect cracks in thin plates over long distances, which could be useful for long-range crack detection in steel bridge members or ancillary structures (Woodward and McGarvie 2000).

This section describes existing NDE/NDT technologies for steel bridge elements that are either readily available for application or sufficiently developed to require limited development for application to steel bridges. These include ET, PT, MT, UT, phased-array ultrasonics, and acoustic emission (AE) monitoring.

12.3.1 Eddy Current Testing

The ET method is a traditional NDE method that has seen widespread use in many industries. The method can be used on any conductive material although many applications are for nonferromagnetic materials such as aluminum, copper, and stainless steel. However, the method can also be applied to ferromagnetic materials such as bridge steel. The magnetic properties of steel introduce complications that need to be addressed in the design of an appropriate sensor. Sensors are commercially available that are designed specifically for application to ferromagnetic materials, particularly the metallurgically complex areas in and around a weld.

The ET method is noncontact and provides real-time test results. The method relies on the interaction of EM fields with the surface of a test specimen, and these fields are capable of penetrating many common coatings to detect cracks that may be concealed from visual detection. Because the method works through coatings, coating removal typically required for MT, PT, and UT is not necessary. Eddy current inspection systems can be small, portable instruments that are relatively inexpensive. Additionally, training and certification is available commercially. These reasons make the ET method suitable for field testing of bridges (Washer et al. 2000). The limitation of the technology is that it only detects surface-breaking cracks and is ineffective for any type of volumetric flaw (subsurface) such as might be detected using UT. However, the reduced cost and time resulting from not having to remove coatings to conduct the inspections is a significant advantage of the technology over PT, MT, and UT.

12.3.1.1 Theory

Eddy currents are induced when an energized probe coil is placed near the surface of a conductive material. Coils are typically energized with an alternating current (AC) that creates a time-varying magnetic field in the area of the coil. When the coil is placed near the surface of a conductive material, small currents in the surface of the material are generated by magnetic induction. Eddy currents are induced at the surface of a certain magnitude and phase depending on the frequency and current level in the coil and the magnetic properties of the material. Properties such as magnetic permeability and discontinuities such as cracks affect the amplitude and trajectories of the eddy currents and consequently the magnitude and phase of the induced current. The probe senses the magnetic field induced by the currents to produce a complex voltage in the coil, which is monitored by a bridge circuit and presented on a display for interpretation.

A differential probe with bidirectional sensitivity can be effective for use on ferromagnetic materials. This type of probe is capable of detecting surface-breaking cracks in steel plate and welds while minimizing changes in voltage produced by any variations in permeability or conductivity of the material. The magnetic properties of steel can vary substantially due to retained magnetization, inclusions in the metal, and varying chemical composition and grain structure. This is particularly true in the area of welds, where chemical composition and grain structure are complex and varied. These variations in local magnetic properties affect the eddy current magnitude and phase and manifest as a high noise level that can mask defect indications or present many irrelevant signals. Use of a bidirectional sensor reduces these effects because each of the probe coils experiences these changes simultaneously such that little or no signal is generated. The probe consists of two circular coils with axes parallel to the surface and perpendicular to each other. In a differential configuration, a coil compares the area influenced by its own eddy currents to the area influenced by the other coil. In the event that both coils are simultaneously affected by the same material conditions, there will be no signal, because the differential coils each produce the same signals, cancelling each other. Any EM condition that is not common to both coils creates an imbalance that generates a change in the output signal of the sensor. When the sensor is scanned over a crack, the output signal is observed, because the crack interrupts the trajectory of eddy currents in the surface of the specimen differently for each coil. The maximum probe response is observed when the crack is perpendicular to the direction of currents from one coil and parallel to the direction from the other coil. Minimal responses occur when the crack is at a 45° angle from the flow of eddy currents from the coils, because the crack will affect each coil equally and result in minimal change in response.

An additional factor in the inspection of ferromagnetic materials is the relatively small skin depth of the sensors. The skin depth is the depth of penetration of the eddy currents and is generally calculated as the depth of influence of at least 63% of the eddy currents generated in the material and can be calculated from the following equation (Cartz 1995):

$$\delta = \frac{1}{\sqrt{\pi f \mu_0 \mu_r \sigma}} \tag{12.1}$$

where

f = frequency (Hz)
δ = skin depth (m)
μ_0 = permeability of free space (Henry/m)
μ_r = relative magnetic permeability of material under test
σ = conductivity of material under test (Siemens/m)

For nonferromagnetic materials, the relative permeability is close to 1, and some depth of penetration can be achieved. For ferromagnetic materials such as bridge steel, the relative permeability is nonlinear and depends on the level of magnetization, and values can range between 1 and several thousand. For the frequencies typically employed, which are in the kHz regime, the resulting skin depth is typically only a fraction of a millimeter and consequently the method will only reliably detect cracks that are surface breaking.

Signal shape is a function of the nature of the defect, and it is influenced by factors such as test material conductivity, magnetic permeability, geometry of the defect, and orientation of the defect relative to the probe. The signal magnitude varies as a function of defect depth; however, it does so in a nonlinear fashion that is complex and difficult to use for estimating the severity of a defect (Washer et al. 2000). The signal magnitude is commonly used primarily to sort very shallow defects, such as scratches in the surface, from actual cracks. Factors such as lift-off variation, probe canting angle, and proximity to geometric changes in the specimen under test also affect the shape and magnitude of the signal. Figure 12.2 shows the probe response for a differential eddy current probe resulting from EDM notches 0.15 mm wide with depths of 0.2, 0.5, and 1.0 mm. The figure shows the reactance (XL) and resistance (R) components of the complex signal generated by the sensor. As shown in the figure, amplitude of the signal response increases with increasing depth for shallow notches. This effect diminishes as the depth increases, such that for notches with depths greater than ~3–5 mm deep, amplitudes may not increase substantially. As a result, the technique can separate very shallow flaws from deeper flaws but is generally not effective for measuring the actual depth of crack.

Inspections are conducted by placing the probe in contact with the specimen and scanning longitudinally along a weld. The method is referred to as noncontact because the coils themselves do not come in contact with the steel. The probe has a plastic wearing surface to protect the coils, and there may also be a coating on the specimen being inspected. However, placing the probe directly on the specimen under test minimizes lift-off distance and provides guidance for the inspector to ensure the lift-off remains constant throughout the test.

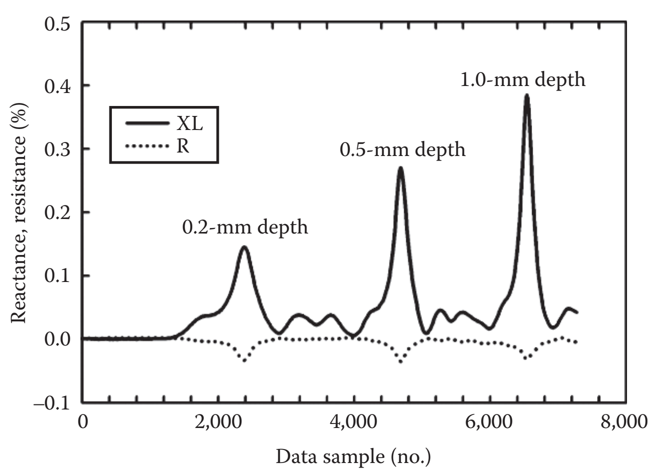


FIGURE 12.2 Response of eddy current sensor to Electrical Discharge Machining (EDM) notches of different depths.

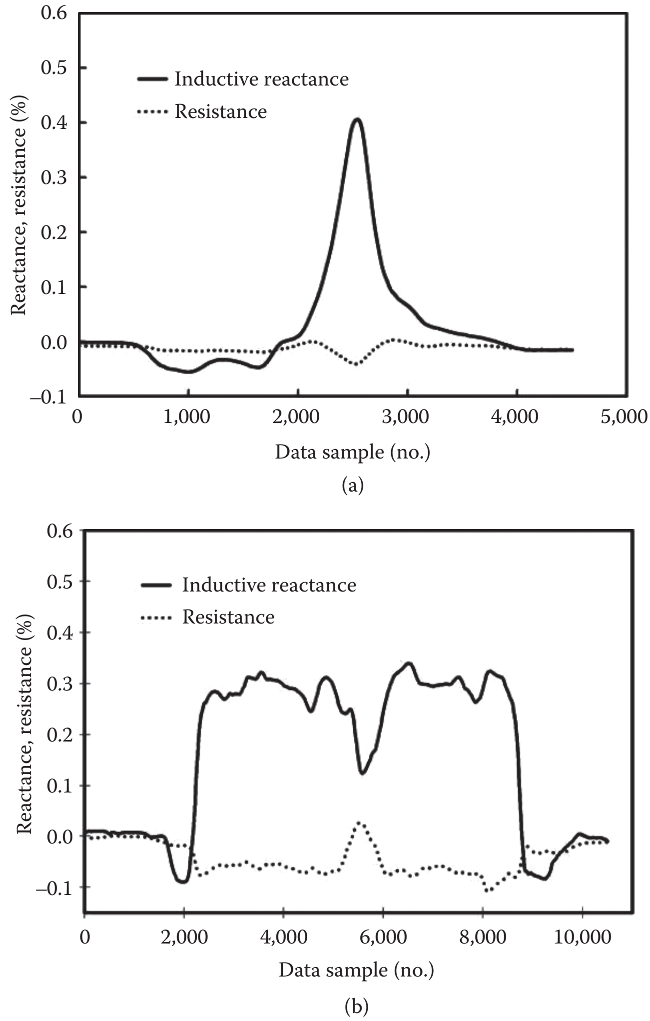


FIGURE 12.3 Eddy current response for (a) a transverse and (b) a longitudinal crack in a weld.

Figure 12.3 demonstrates the probe response to a transverse crack in the crown of a weld (Figure 12.3a) and a longitudinal crack in a weld toe (Figure 12.3b). As shown in the figure, the probe response is similar to that generated by the EDM notch for the transverse crack. These signals are generated as the probe is scanned longitudinally along the weld surface. As a result, the length of the longitudinal crack is represented by the length of the displaced signal shown in Figure 12.3b. The horizontal axis shown is data samples, but could be represented as time, since data samples are collected at a fixed time interval. If the scanning rate is constant, this axis could be distance along the weld axis. The signals shown in Figure 12.3 are typical of ET signals that are interpreted by the inspector to identify a crack.

12.3.2 Dye Penetrant Testing

Penetrant testing (PT) is one of the simplest and most widely used technologies for crack detection in steel bridges. The method consists of cleaning the surface of the steel and grinding to remove anomalies in the surface (e.g., rough welds) (ASTM 2011). Dye is applied to the surface and allowed to rest on the surface for a prescribed dwell time, typically 5 minutes or more. This surface is again cleaned to remove all of the dye applied to the surface; dye that has entered a surface-breaking crack is not removed by the cleaning process. A developer

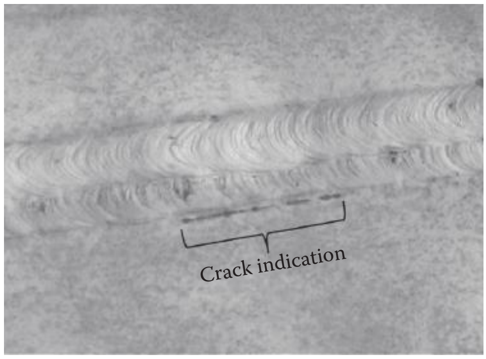


FIGURE 12.4 Photograph of crack indication at the toe of a fillet weld.

is applied to the surface of the steel, typically a white spray-on powder that provides a color contrast with the dye. Dye will emerge from the crack through capillary action, resulting in an indication that is observed by the inspector and interpreted as a crack. Figure 12.4 shows a crack indication along the toe of a weld.

The process of cleaning, apply dye for a specified dwell time, and then applying developer and waiting for dye to emerge from a crack is time consuming and requires removal of coating and complete cleaning of the surface. If the surface is not adequately cleaned, interpreting results can be very difficult, because dye remains after the cleaning process that bleeds through the developer and can create false indications or make true indication difficult to see and interpret effectively. On very rough surfaces, such as the irregular surfaces of field welded or poorly welded connections, adequate cleaning of the dye is difficult, and as a result, there can be numerous noncrack indications, and bleed-through of the dye can make interpretation of results very challenging. Extensive grinding may be required to make the surface sufficienlty smooth that adequate cleaning of the dye can be accomplished.

12.3.3 Magnetic Particle Inspection

MT is a commonly used technology for the detection of cracks in steel. This technology is commonly used in steel fabrication shops and other industrial environments. The technology is relatively simple and rapid to apply, and industrial standards exist that provide suitable procedures for implementation (ASTM 2008). MT is commonly used for the inspection of fillet welds during the fabrication of steel bridges to detect transverse and longitudinal cracks.

The technology works by inducing a magnetic field in a ferromagnetic material (steel) between opposite magnetic poles, establishing a magnetic circuit including the steel between the poles. Sudden geometric changes in the steel result in field leakage at the surface of the steel. Finely divided iron particles are applied to the area between the poles, and if there is magnetic field leaking from a crack or crack-like defect, the iron particles become entrapped in the leaking magnetic field and form an indication. Iron particles typically contain a dye such that the particles are a contrasting color with the surface of the material being inspected, typically either red, gray, or yellow color. This technique is suitable only for surface-breaking cracks or very near-surface cracks. An example of a surface breaking crack indication is shown in Figure 12.5.

Hardware typically employed in the field utilizes AC EM yokes to induce the magnetic field in the steel. The alternating magnetic field that results from AC in the electromagnet provides *particle mobility* that allows the iron particles to more readily move to the area of magnetic field leakage or be more easily displaced from the surface using a blower if no leakage is present. These AC yokes are usually designed to be energized using conventional 60-Hz, 120-V power sources or may be purchased in battery-operated units.

The MT technique can also be applied using permanent, rare-earth magnets. Relatively inexpensive permanent magnet assemblies are available for approximately $500 that can be easily transported and stored within an inspection van and utilized to find cracks in steel bridges or to confirm suspected cracks discovered during visual inspections. As shown in Figure 12.6, the magnets are mounted in cylindrical

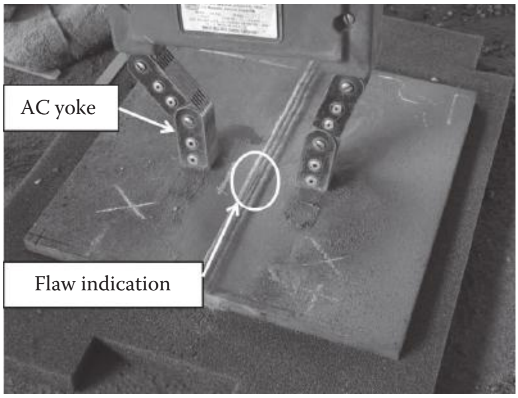


FIGURE 12.5 Typical magnetic particle testing using AC yokes and yellow particles.

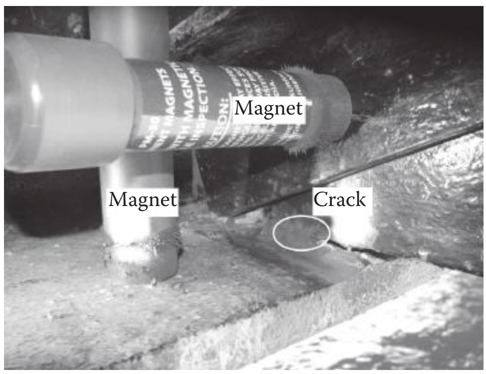


FIGURE 12.6 Photograph of permanent magnet magnetic particle testing application and a typical crack indication.

holders, typically attached together with a wire rope for convenience in storage and movement. The advantage of using this approach is that it is very simple and quick to apply, as compared with PT.

With MT, surface cleaning requirements are less rigorous relative to PT, and indications on irregular surface can be much easier to interpret. As a result, the method can be applied more rapidly for detecting cracking or interpreting crack indications detected visually. A disadvantage of using permanent magnets is some loss in the sensitivity of the technique relative to AC yokes, because the iron particles are less mobile than when AC power is used. The magnitude of the field induced in the steel may be less, relative to power yokes, depending on the geometric configuration of the test.

12.3.4 Ultrasonic Testing for Steel

12.3.4.1 Background

Most acoustic techniques rely on a few basic relationships to detect discontinuities in a material or evaluate elastic properties. Generally, the relationship between the velocity of an acoustic wave and the elastic modulus of a material can be described by the relationship derived from a one-dimensional wave equation, neglecting Poisson's effect:

$$v_l \approx \sqrt{E/\rho} \tag{12.2}$$

where v_l is the velocity of a longitudinal wave, E is the modulus of elasticity, and ρ is the density. This equation is useful for demonstrating that as a material's modulus decreases, the velocity decreases proportionally to the root of the modulus change. This effect is used in many qualitative applications, including pulse velocity measurements in concrete, where reductions in pulse velocity are correlated with damaged or deteriorating concrete.

Reflection-based ultrasonic techniques, such as the detection of subsurface flaws in steel or concrete, depend on the contrast in acoustic impedance between intact material and a flaw or discontinuity. The acoustic impedance of a material is related to the material's density and modulus through the following equation:

$$z = \rho v_l \tag{12.3}$$

where z is the acoustic impedance of a given material. When a wave propagating in one medium interacts with a boundary of a second material of different acoustic impedance, some portion of the wave is reflected and can be subsequently detected and analyzed. The proportion of the wave that is reflected depends on the contrast between the acoustic impedance of the two materials. For example, when a wave propagates through a steel plate and interacts with the back surface of the plate, which is a steel–air interface, virtually all of the wave is reflected. Similarly, if the plate contains an open crack, there will be a steel–air interface that reflects the acoustic wave. This basic principle is used for the detection of subsurface flaws in concrete, steel, and other materials.

UT is one of the most widely utilized tools in the world for the detection of defects and flaws and assessment of material properties. Before the development of modern technologies, rudimentary acoustic techniques were sometimes used in an effort to determine if there were flaws or defects in a weld (Kinzel et al. 1929). In this early method, a stethoscope was placed on the surface of the steel plate near the weld, and a hammer was used to impact the surface of the steel, creating vibrations (acoustic waves) within the material. If there was a major defect or flaw in the weld, the vibrations resulting from the hammer impact would be dampened, and this dampening could be detected by the operator through the stethoscope on the surface of the steel. This basic acoustic technique was replaced with RT and modern ultrasonic transducer technologies developed between 1940 and 1960 (Hellier 2001). UT was first integrated into the bridge fabrication process in 1969 with the publication of the American Welding Society (AWS) Specification for Welded Highway and Railway Bridges, which included an appendix describing the UT of groove welds (Shenefelt 1971). The modern version of this code, AWS D1.5, Bridge Welding code, includes essentially the same procedure as that introduced in 1969 for the fabrication inspection of welds (AWS 2010).

The procedures described in the bridge welding code are intended to ensure good workmanship and quality during the fabrication of steel bridge members and include criteria for identifying subsurface indications based on the amplitude-response created using a shear wave transducer scanned over the surface of the steel. Indications within the weld are reported according to the reflected amplitude of waves, compared with a 0.06-in. side-drilled-hole calibration sample. The length of an indication is determined by an amplitude-drop method that evaluates the linear distance over which the reflected amplitude from the indication is equal to or greater that 50% of a specified value. These procedures are intended for the fabrication of bridges, to identify poor workmanship or defective welds, such that repairs can be made in the shop. As such, the procedures are intended to identify subsurface weld flaws such as lack of fusion, porosity, slag inclusion, and cracks, but do not necessarily assess the severity or criticality of particular flaw or indication. These procedures are sometimes used in the field to evaluate welds and weld quality and fatigue crack detection. In the field, UT has been used more widely for specialized applications, usually for circumstances where access restrictions are severe. Applications include flaw detection in trunnion shafts, anchor bolts, and bridge pins; the latter is described in Section 12.3.4.2.

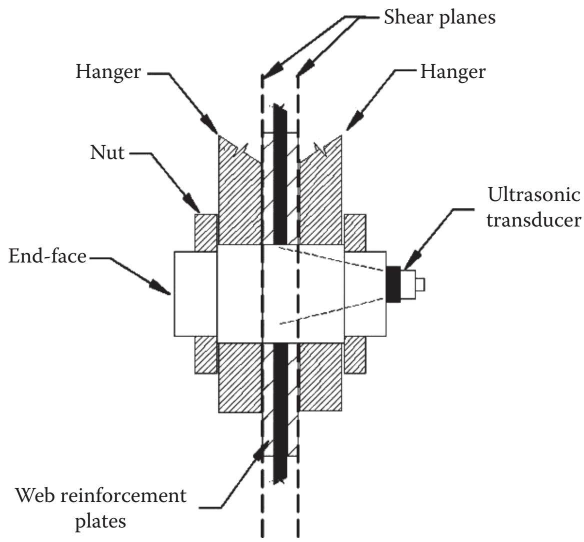


FIGURE 12.7 Schematic diagram of a pin and hanger connection showing ultrasonic transducer position.

12.3.4.2 Crack Detection in Pins

The collapse of the Mianus River Bridge in June 1983 indicated the susceptibility of pin and hanger connections to suffer corrosion-related damage and catastrophic collapse. Pin and hanger connections are typically designed to provide thermal expansion and contraction of a bridge and are normally located beneath an expansion joint, making them susceptible to corrosion damage. The load path of the connection creates two shear planes within each pin, one at each of the intersections of the web reinforcement plate and the hanger plate as shown schematically in Figure 12.7. If a pin were to fail along this shear plane, the suspended span can be unsupported and collapse. As a result, detection of cracks that may occur at this plane is critical to ensure bridge safety; however, there is no access to this area for visual assessment.

As shown in Figure 12.7, only the flat ends, or end-faces, of the pin are available for inspection in a typical pin and hanger connection. To detect a crack in the pin along the shear plane, ultrasonic transducers are scanned over this surface. Flat transducers with frequencies ranging from 2 to 5 MHz are commonly used as well as angle-beam transducers with angles between 10° and 15°. These transducers are used to launch longitudinal waves into the pins that will reflect off the surface of cracks along the shear plane. Calibration specimens that match the general dimensions of the pin, and include saw-cuts or EDM notches located in the critical shear-plane regions, are often produced for use during field testing (Graybeal et al. 2000; Moore et al. 2004; Januszkiewicz 2010).

Angle-beam transducers are sometimes used to improve the ability of the UT to detect defects initiating at the surface of the pin barrel. Pins typically have a shoulder where the threaded portion of the pin intersects with the pin barrel. This shoulder is normally concealed beneath a large nut and not available for inspection. Using a slightly angled beam will enable the center of the beam profile, where waves amplitudes are largest, to interact with the barrel surface where cracks may be anticipated. When using straight-beam transducers, beam spread must be relied on, and this may result in lower sensitivity, since the center of the beam profile will not intersect with the crack.

12.3.4.3 Thickness Gauges

A common problem encountered in the inspection of steel bridges is corrosion of the steel that results in loss of section. Atmospheric corrosion rates are relatively low in most environments, but when drainage patterns or design details expose steel to sitting water or run-off from bridge decks, corrosion damage can be significant. The amount of section loss or thinning of steel plates can be difficult to quantify visually, and access for measurement by calipers is often limited or not possible due to geometric constraints. Ultrasonic thickness gauges are a widely available tool for determining the thickness of metal plates and coatings. This technology works by launching an ultrasonic wave and measuring its time-of-flight as it propagates through the material, reflects off the opposite side, and is detected by the transducer.

The typical test arrangement is shown in Figure 12.8.

Calibration standards of known thickness and of the same or similar material to the material under test are used to calibrate instruments in the field, such that the instrument reads a digital thickness measurement (Fowler et al. 1997). This technology is widely available, low cost and easy to operate, and is currently utilized by bridge inspection teams in many states. Generally, no certification and only minimal training is required to implement this technology. The use of ultrasonic thickness gauges has been recommended by the FHWA following the collapse of the I-35W Bridge in Minneapolis, MN, to assess the remaining section in gusset plates as part of a capacity analysis (2010).

Ultrasonic thickness measurements are conducted from one surface of the steel to measure the thickness of the steel plate. Coatings and/or corrosion on the surface of the steel need to be removed to achieve the most accurate measure of thickness; when left in place, coatings or corrosion on the inspection surface can result in an increased estimate of the plate thickness (Damgaard et al. 2010). Coupling of the transducer to the surface can be difficult for corroded surfaces where there is pitting and section loss, and grinding of the surface may be required to achieve adequate coupling (Cella 1994).

A disadvantage of ultrasonic thickness measurements is that the measurements are spot measurements, that is, only the thickness of the plate directly below the transducer is assessed. These measurements can be very time consuming, with measurement patterns of one measurement every 3 or 6 in. or more are used to provide an assessment of the overall condition of a plate. Under this scenario, localized areas of severe corrosion and pitting may lie between the measurement locations, leading to an underestimate of the most severe section loss in the plate.

New technology implementing a scanning approach that can quickly record and store many more thickness measurement at much smaller spacing has become commercially available and practical for field use. This technology could improve the reliability of inspections by providing more complete and accurate data on section loss in steel members. This technology utilizes "B-Scan" data recording that determines the thickness of the material along a scanning line. Figure 12.9a shows an inspector using a scanner to make thickness measurements on a gusset plate. Results are typically presented as a color-coded two-dimensional (2D) image showing the distance along the horizontal axis and the thickness of the plate along the vertical axis, known as a B-scan, as shown in Figure 12.9b. As shown in the figure, thickness measurements appear as different heights in the image, and the thicknesses can also be color-coded. The scanned data is stored in memory and can be downloaded for archiving.

The advantages of B-scan technology includes increased coverage that can be obtained, and reduction in the inspection time is required. As compared with conventional UT thickness measurements, where measurement locations and values may be recorded manually, scanning technologies store

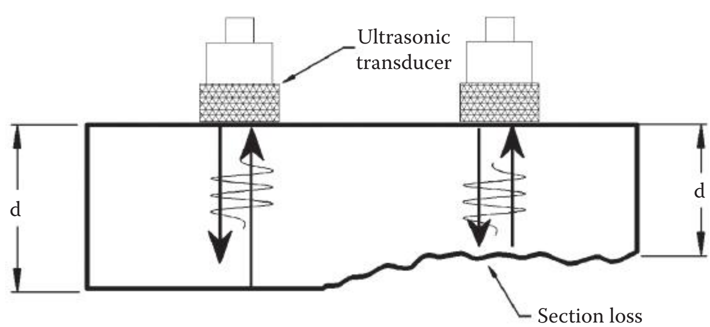
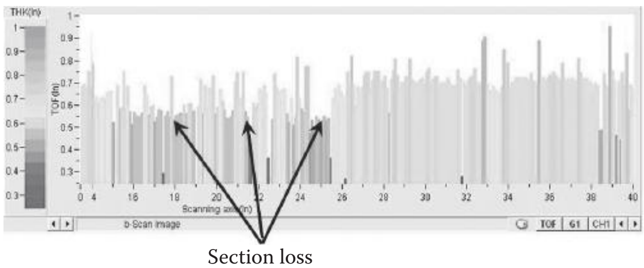


FIGURE 12.8 Schematic diagram of an ultrasonic thickness gauge.

(a)



(b)

FIGURE 12.9 (a) Photograph of B-scan ultrasonic thickness measurement and (b) B-scan data output. (Images courtesy of Physical Acoustics.)

measurements rapidly to the onboard computer memory such that only the orientation and location of the scan lines need to be recorded manually. This technology is common and standard in many industries where corrosion in pipes, pipelines, and storage tanks is common. Dry-coupled transducers as well as traditional couplant transducers are commercially available. Additional surface cleaning is typically required, because a larger area of plates is assessed. However, the technology is sometimes implemented without removing paint, if the paint is consistent in thickness and well bonded to the surface.

12.3.5 Phased Array Ultrasonics

Another ultrasonics-based technology that can be used for improving bridge inspection reliability is phased-array UT technology. This somewhat advanced technology is suitable for specific inspection challenges, such as the inspection of bridge pins or other unique geometries where access limitations are severe, as well as weld scanning. Phased array technology has been used in the medical industry for many years, but has only recently been developed as a practical industrial inspection tool. The FHWA first tested the application of phased arrays for crack detection in bridge pins in 2001 in collaboration with the Edison Welding Institute (Lozev 2001; Washer and Fuchs 2001; Lozev et al. 2002). Since that time, commercially available equipment has become much more portable and much more cost-effective. While an advanced technology requires significant training to implement, ultrasonic phased array has some unique capabilities that make it a suitable technology for unique and challenging inspection scenarios.

Phased arrays use the same transducer technologies and physics of any UT device. However, in a phased array, many transducer elements are used in coordination to enable the waves generated in the test material to be focused at a particular depth or steered in certain directions (angles). Wave physics, primarily constructive and destructive interferences, are controlled by sophisticated computer programs that enable the accurate timing of pulses such that the waves are launched in a *phased* manner, enabling the control of the resulting wave propagating in the material. This control allows for beam steering, scanning over a volume from a single location, and dynamic focusing at different depths in the material without changing transducers (Roth et al. 2010).

Signal response from defects and other internal features are typically displayed in color-coded images that illustrate the location (e.g., depth in a plate) and the amplitude of the received waves. These images are visually appealing, and the steering characteristics of the transducer can improve test efficiencies and enable more comprehensive coverage of certain geometries than could be achieved using transducers with fixed incident angles.

Practical applications in the field have been focused primarily on bridge pins and other unique geometries that can utilize the unique beam-steering characteristics of a phase array device. Training requirements are significant for a phased array device to address the unique characteristics of the images produced from the testing. Standard methods of interpreting results that capitalize on the unique characteristics of a phased array have yet to be developed, such that application of the technology lacks standardization.

An example application is shown in Figure 12.10 for a fatigue crack in a steel eyebar. The surface of an eyebar near the bore, where cracks are likely to occur, is typically inaccessible for visual inspection due to adjacent eyebars linked along the same pin. Using traditional ultrasonic methods is challenging, because the surface of the eyebar on which the transducer is placed is irregular due to manufacturing processes and damage that frequently occur. As a result, coverage of the bore area of the eyebar where cracking is likely to occur can be uncertain. However, using a phased array and sweeping the beam in an angular fashion, which is done automatically by the instrument and associated transducers, mitigates this issue. Even if surface variations occur, adequate coverage of the bore area can be achieved because the beam has an angular sweep. In Figure 12.10, the output of a typical phased array is shown, including the A-scan, the traditional ultrasonic sensor output, a sectorial image that shows the sensor response over the swept angle, and a B-scan, which shows essentially the same information as the sectorial scan, but with geometric adjustment to account for the angular sweep (Lozev et al. 2002).

Advantages of phased array ultrasonics include increased area coverage and its ability to be used on members with more challenging geometries. Its disadvantages include difficulty in signal interpretation, higher cost than other ultrasonic methods, and a lack of standard procedures available for tests using this method. The technology would typically be implemented by specialists and only for

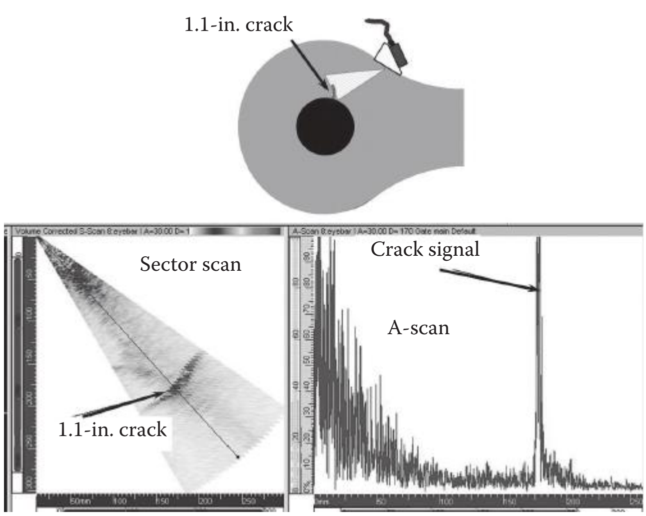


FIGURE 12.10 Phased array images of a fatigue crack growing from an eyebar bore.

unique inspection challenges where access limitations are severe. These applications include bridge pins, trunnions, or eyebars. Phased array may also be utilized in cases where critical subsurface defects need to be fully characterized beyond the capabilities of traditional UT to support fracture mechanics, such as a transverse butt weld in a fracture critical member.

12.3.6 Acoustic Emission Technology

AE is a method of detecting the onset of damage in materials based on burst of elastic energy associated with the formation of the damage. The technique was first developed in the 1950s by materials scientist exploring the formation of the microstructures in metals and later developed as a means of monitoring the development and propagation of the damage due to static and fatigue loading (Scott 1991). Since that time, AE testing has become common for testing pressure vessels, aerospace vehicles, and other engineering applications. More recently, AE methods have been developed exploring the application of AE as an NDE method for concrete and concrete structures, and these are discussed in Section 12.4.4.

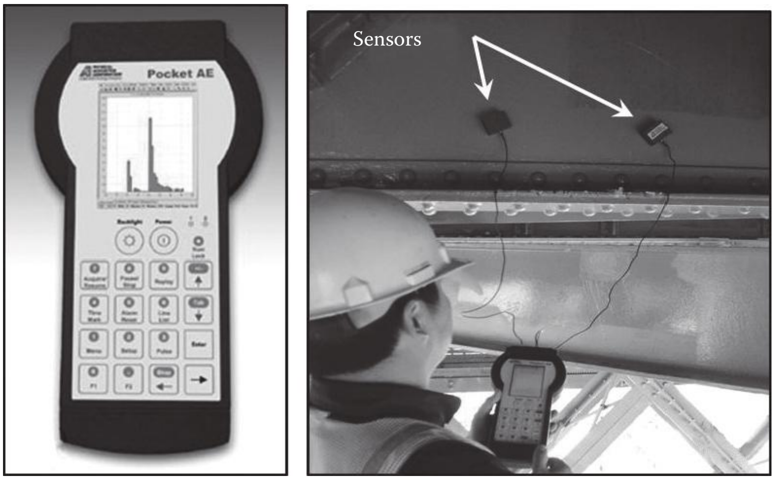
The fundamental theory behind the generation of AEs in metals is that propagation (growth) of a crack releases a small burst of elastic energy caused by the extension of the crack surface on an atomic level and plastic-zone development processes surrounding the crack tip. This burst of elastic energy propagates as an acoustic pulse through the material and can be detected by sensors coupled to the surface of the material under test. The AEs are typically discriminated from other noise that may be present, such as traffic noise on a bridge and rubbing of bearings, based on waveform characteristics (Nair and Cai 2010). Analysis of monitoring results typically consists of assessing the number of AE events per unit time, with increased AE activity being associated with crack nucleation and growth.

AE has traditionally been implemented for bridges as a monitoring technology, with a number of sensors placed permanently on a structure to monitor an area for incipient crack growth (Hutton and Skorpik 1978; Vannoy et al. 1987; Clemena et al. 1995). Monitoring the AE activity of known cracks or the effects of retrofitting on arresting crack development and growth are the most typical applications, as opposed to monitoring a bridge with no known cracks (Prine and Marron 1997; Kosnik et al. 2010). Monitoring systems for AE testing typically consists of multichannel (16+ channels) systems that can be mounted in the field and communicate data through phone lines.

An innovative new technology has been recently introduced to the market that has the potential to provide a new tool for inspectors: a small, two-channel AE data acquisition system that can be used to monitoring the crack growth activity over a short period of time. This technology could be implemented to assess the stability of a crack detected by visual inspection, by evaluating the level of AE activity at the crack during a normal inspection to determine if the crack is growing under applied loading. Such information could help an inspector to identify a rapidly growing crack, even if its length has not yet reach the established criterion, or identify a crack that is not growing. An example of a handheld AE collection device is shown in Figure 12.11a and its application to a steel bridge girder is shown in Figure 12.11b. As shown in the figure, sensors are attached using magnetic holders to the surface of the steel, and data is recorded on a handheld device. More traditional applications of the technology include a data-collection station hard-wired to a number of sensors and mounted on the bridge, with data monitoring performed through telecommunication links.

12.4 NDE Technologies for Concrete Bridge Components

The primary damage mode affecting concrete structures is corrosion-induced damage, particularly delamination and spalling resulting from expansion of embedded reinforcing steel. When steel corrodes, volumetric expansion of the steel occurs as iron is converted to various iron oxides. This expansion can be four to six times the original size, and the resulting tensile stresses in the concrete surrounding the reinforcing steel result in horizontal cracks at the level of the rebar known as delaminations. Freeze-thaw damage and cracking also affect the performance of concrete bridge elements. Corrosion damage to prestressing steel is an important damage mode that afflicts prestressed and posttensioned concrete bridge elements.



(a) (b)

FIGURE 12.11 (a) Handheld two-channel acoustic emission (AE) instrument and (b) application of the AE sensor on a steel bridge. (Image courtesy of Mistras Corp.)

This section discusses a number of NDE/NDT technologies applicable to the condition assessment of concrete bridge elements, including ultrasonic pulse velocity (UPV), IE, GPR, AE for concrete, radiography, IR thermography, and magnetic flux leakage (MFL).

12.4.1 Ultrasonic Pulse Velocity

UPV measurements are a common method of assessing concrete quality and strength. The method consists of measuring the time-of-flight of an ultrasonic wave propagating through the concrete of known thickness, to determine the pulse velocity in the material between the transmitting and receiving transducers. A combination of the amplitude and travel time of the acoustic wave can also be analyzed to detect defects or discontinuities in the material, either by reflections created by the internal defects, loss of signal amplitude received, or by the effect of the defect on the apparent wave velocity (Thomas et al. 1997; Bungey et al. 2006).

The velocity, v, of an acoustic wave is generally proportional to the elastic modulus (E) and the density of the material according to Equation 12.2. As such, a stiffer material generally propagates an acoustic wave at a higher velocity than a softer material of a similar density. When pulse velocities differ over different areas measured in the same member, it can indicate varying concrete quality, that is, variations in the stiffness of the material in situ (Dilek 2007). Lower velocities may be attributed to lower strength, poor compaction, physical or chemical deterioration, inadequate curing, or damage and cracking as a result of structural defects (Thomas et al. 1997).

Aggregates in the concrete scatter high-frequency ultasonic waves, and consequently low-frequency pulses, typically with frequencies between 20 and 250 KHz, are used (Thomas et al. 1997; IAEA 2002; ASTM 2009). Typical commercially available transducers operate at 54 kHz. An example of typical equipment for UPV is shown in Figure 12.12.

UPV is a simple and portable technology that requires minimal training, and equipment is readily available in the commercial market. UPV measurements are sometimes employed for new construction to evaluate areas of understrength or poor quality concrete and voids or honeycombing due to construction errors (Dilek 2007). For the inspection of in-service bridges, the technology can be used to better define the extent of corrosion-related damage or test the integrity of concrete components such as substructure elements or open concrete beams. The technology can also be used to assess the effects of accidental fire or chemical attack on concrete strength or provide estimates of the surface crack depths (IAEA 2002; Ghorbanpoor and Benish 2003).

FIGURE 12.12 Photograph of ultrasonic pulse velocity equipment.

A disadvantage of the technique is that it generally requires access to both sides of a member, such that direct transmission of acoustic waves from the transmitter to the receiver can be obtained. Indirect measurements, in which the transmitter and receiver are on the same surface, are generally more difficult to interpret. To use the method to determine in situ material properties of concrete, velocity measurements in samples of the same material (with known properties) are usually needed to correlate results and obtain quantitative data on material properties. Another disadvantage is that UPV is a point-testing technique, as the test only acts on the concrete between the transducers. As a result, the application of the method to a large area of concrete can be time consuming.

Methods to generate three-dimensional (3D) and 2D tomographic images using UPV measurements have also been developed (Pla-Rucki and Eberhard 1995; Buyukozturk 1998; Martin et al. 2001). Images are created from multiple velocity measurements, typically using a single transducer to launch a wave and several receiving transducers. Research has been performed on the use of array systems that use multiple sensors to generate a 3D tomographic image of the interior structure of the concrete (Buyukozturk 1998; Rens et al. 2000; Krause et al. 2001; Delahaza et al. 2008; Friese et al. 2010). The technology has also been applied for imaging internal posttensioning tendons in concrete bridges (Martin et al. 2001; Wiggenhauser and Reinhardt 2010).

Another example of ultrasonic imaging in concrete is a recently developed system known as MIRA (Ultrasonic Shear Wave Test Method), which uses a phased array of dry-point ultrasonic transducers to produce tomographic images of concrete structures in both 2D and 3D. This device has been used to successfully detect flaws in grouted posttensioned cable duct systems (Kohl et al. 2003; Streicher et al. 2006; Delahaza et al. 2008; Wiggenhauser and Reinhardt 2010).

An advantage of tomographic imaging is that it presents visual results indicating areas of damage or voids in two or three dimensions. The arrays used to generate tomographic images are sometimes not significantly more difficult to set up and use than is UPV equipment, depending on the application and type of array used (Daigle et al. 2005). However, interpretation of results can be very difficult.

Various other techniques that apply the general relationship between acoustic velocity and concrete quality used in UPV have been developed, particularly for bridge decks. These include various sonic-echo techniques that use either a transducer or some form of impact to assess concrete by characteristic wave velocities or attenuation of waves. Among the most widely used and well developed is the IE method described in Section 2.3.2.

12.4.2 Impact Echo

IE is also a well-established NDE technology typically applied for the condition assessment of concrete decks and pavements. This technology can also be applied for the detection of subsurface defects in bridge soffits, parapets, and substructure elements. For example, the technology could be applied for the detection of delaminations in the early stages, before concrete spalling occurs, in critical areas such as soffits of overpass bridges or substructures adjacent to roadways, where spalling concrete may be a safety concern due to its impact on traffic.

IE is based on generation of transient elastic stress waves by an impact, typically a small steel sphere. Low-frequency stress waves are generated by the impact and propagate through the concrete, being reflected by the boundaries of the concrete, for example, the bottom of a deck, and/or by defects within the concrete, such as delaminations. The response signal is detected by a transducer placed adjacent to the location of the impact. Results are displayed in the frequency domain as plots of amplitude versus frequency or spectra (Wouters and Poston 2001; Shokouhi et al. 2006; Tinkey et al. 2008). Resonances that appear in the frequency plots are used to evaluate the thickness of a concrete component, or if a subsurface defect exists, the wave path is interrupted and resonance established in accordance with the depth of the defect (Sansalone and Carino 1998; Shokouhi et al. 2006; Tinkey and Olson 2007). In that case, higher-frequency amplitude peaks are sometimes observed in the Fast Fourier Transform (FFT) display, which indicates the depth of the defect within the material. Under certain circumstances, when the delamination is thin and covers a large area, a lower frequency may be produced (Zhu and Popovics 2006). The equipment used in this test typically consists of a data acquisition system, handheld transducer unit, spherical impactors, and accompanying software and computer packages, as shown in Figure 12.13.

IE can be used to detect delaminations in concrete components including decks with or without overlays (Sansalone and Streett 1998; Shokouhi et al. 2006; Gucunski et al. 2008; Gucunski et al. 2010). Other applications include the characterization of surface opening cracks, concrete pavement or overlay thickness, and the detection of voids in grouted tendon ducts (Krause et al. 2002; Tinkey and Olson 2007; Limaye 2008).

Handheld scanning devices have been developed to automate the process and increase the speed of IE tests, and these are commercially available. These scanning devices take automated measurements at predefined intervals, such that an area can be assessed rapidly by simply running the device over the surface of the concrete. B-scan images of results can be produced that improve interpretation capabilities (Schubert et al. 2004). Cart-mounted and vehicle-mounted systems are also under development to allow for IE method to be applied more rapidly for bridge decks. Research has also been conducted into the use of air-coupled IE to make measurement more rapid (Zhu and Popovics 2006; Kee et al. 2011).

A significant advantage of the IE technique is that access to only one surface of the structure is required, as opposed to UPV, which generally requires access to two sides. As a result, the technique is very versatile and can be applied to a wide variety of structural shapes, including concrete box beams, soffit areas of concrete, abutments, and concrete decks.

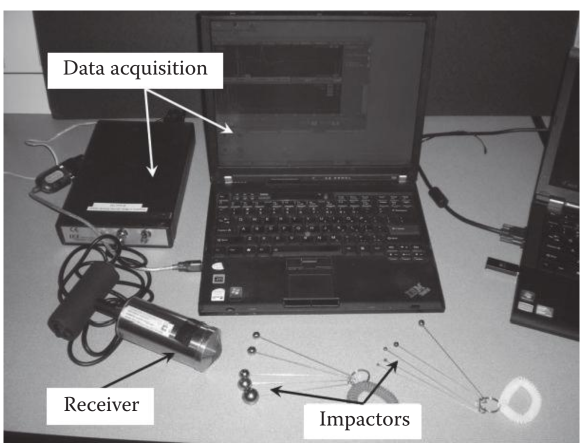


FIGURE 12.13 Photograph of impact echo equipment.

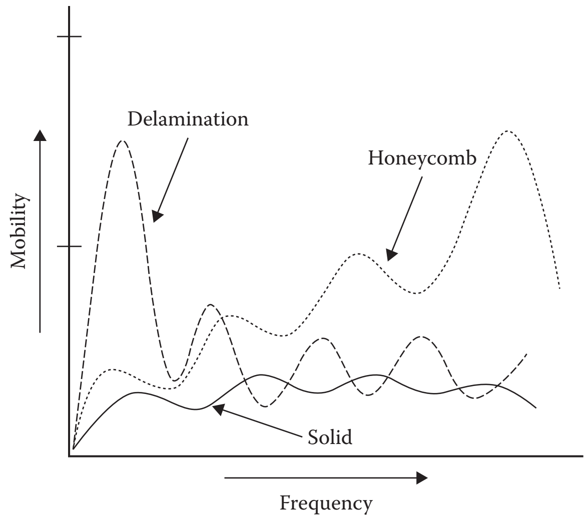


FIGURE 12.14 Example mobility plot for impulse-response.

Disadvantages of the technique include the time required to scan a large area and the hands-on access to the surface that is required to implement the technology.

A method that is related to IE is the impulse-response method, which has been used traditionally for the assessment of foundation elements (Davis 1994). In contrast to the IE method, which impacts the surface of the concrete with a small metal sphere, impulse response uses a ~2-lb instrumented sledge hammer to impact the surface of the concrete, generating stress waves in the concrete ~100 times that of the IE method (Davis and Peterson 2003). A velocity transducer (geophone) placed on the surface records the response of the concrete to the impact. The data from the load cell in the hammer and the detected response from the velocity transducer are combined to determine the "mobility" of the concrete in response to the impact. Plots of the "mobility" as a function of frequency in response to the impact are used to assess the presence of anomalies in the structure, such as subsurface honeycombing, delaminations or voids beneath the surface. An example mobility plot is shown in Figure 12.14, which shows mobility plots for a solid concrete structure and the effect of honeycombing on the higher-frequency components of the mobility plot and delamination on the lower-frequency components.

The impulse response technology provides a general condition assessment that is applicable to large structural components like piers and abutment to assess subsurface anomalies, such as voids, honeycombing or delaminations, or degraded concrete resulting from Alkali-Silica Reaction (ASR) or other effects. The method has also been used to assess integrity in adjacent box girder bridges to identify deteriorated members and assess the condition of transverse ties (Davis and Peterson 2003; Clausen and Knudsen 2008). The method is often coupled with methods with more precision, such as IE, to identify areas where further investigation is warranted. The advantage of the technique is that it is relatively simple to apply, evaluates a large concrete component from a limited number of tests, and is rapid. The primary disadvantage of the method is that the results do not provide much dimensional information about a detected anomaly; secondary methods are needed to determine the dimensions of a detected anomaly.

12.4.3 Ground-Penetrating Radar

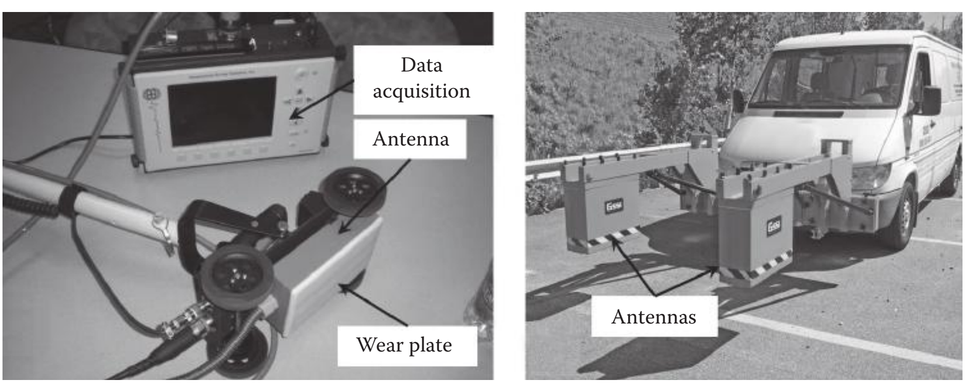
GPR is a method of assessing the condition of concrete bridge decks and other concrete bridge elements. The method is based on the propagation of EM waves in dielectric (nonconductive) mediums such as concrete. When the waves encounter a boundary between materials that have different dielectric constants—for example, the boundary between concrete and air—a portion of the wave energy is reflected at the boundary (Warhaus et al. 1993; Cardimona et al. 2001; IAEA 2002; Maser 2008). When the wave encounters a conductor, such as rebar in concrete, the wave is entirely reflected. Changes in the amplitude and phase (i.e., velocity) of EM waves launched into the concrete and returning to a receiving antenna are interpreted to determine areas of concrete that are deteriorated (IAEA 2002). Operating frequencies for GPR antennas used for concrete deck inspections typically range from 900 MHz to 2.5 GHz.

GPR systems are generally configured as either ground-coupled or air-coupled devices. Ground-coupled devices utilize the antenna held against the surface of the concrete and are typically equipped with wear plates to protect the antennae and to allow the units to be dragged along the surface of the concrete, as shown in Figure 12.15a. Air-coupled devices utilize antennae not in contact with the surface being tested and can be vehicle-mounted or pulled in a trailer, as shown in Figure 12.15b. The advantage of air-coupled systems is that the testing can be conducted at driving speeds, allowing for larger deck areas to be inspected more quickly. Ground-coupled units allow more energy to be imparted into the material under test and typically have reduced air-surface signals. Because the surface of the antenna is in contact with the material under tested, the velocity at which the antenna can be moved is limited.

The method by which GPR detects deterioration within concrete is by interpreting the reflected EM waves detected by the antennae and assessing variations in the response that imply internal deterioration such as corrosion damage or voids (Parrillo et al. 2006; Maser 2009). Data is typically interpreted on a radargram that displays multiple waveforms in a waterfall plot (B-scan) that consists of multiple signals aligned with signal amplitudes represented on a gray scale (Maser and Roddis 1990; Maser 2008, 2009). Strong signal responses from the reinforcing bars (rebar) appear as hyperbola-like features in the data as the antenna is moved over the transverse rebar. The loss of signal amplitude and increased time delay for the rebar signal indicates deterioration in the concrete and can result from elevated chloride content, deterioration of the concrete, and/or corrosion of the rebar.

GPR can be used for several applications including determining concrete thickness in cases where only one side of the concrete is accessible, concrete cover measurements for Construction Quality Control (QC) or Quality Assurance (QA), or identification of in-service bridges where early corrosion issues may be anticipated (Maierhofer 2003). GPR has also been used to locate tendon ducts within concrete bridge decks and/or beams (Maierhofer 2003; Maierhofer et al. 2004).

Although traditionally applied to bridge decks, handheld GPR devices can also be applied to other bridge superstructure and substructure components. As with IE, this technology in its handheld form could be utilized for the assessment of corrosion damage in bridge soffits and substructure elements. Handheld units like that shown in Figure 12.15a are typically equipped with encoders to track the position of the units and generate the associated radargram.



(a) (b)

FIGURE 12.15 Photographs of (a) ground-coupled ground-penetrating radar (GPR) antenna and (b) air-coupled, vehicle-mounted GPR antennas. ((b) Image courtesy of GSSI, Inc.)

A significant advantage of GPR is its capability to evaluate concrete bridge decks with asphalt overlays and to provide reasonably accurate percent deterioration estimates (Cardimona et al. 2001; IAEA 2002; Maierhofer 2003; Parrillo et al. 2006). It can be rapidly applied to the bridge and causes relatively little traffic disruption when air-coupled antennae are used.

There are some disadvantages to this technology. Research has indicated that GPR results can be ambiguous (IAEA 2002; Shin and Grivas 2003; Barnes and Trottier 2004). Training is required to interpret the signals and images generated. GPR is typically used to detect concrete deterioration due to corrosion; GPR is not effective for detecting very thin, dry delaminations or debonding (IAEA 2002; Maser 2009; Gucunski and Nazarian 2010; Gucunski et al. 2010). Recent studies have shown that combining GPR with IR thermography, which has improved capability to detect dry delaminations, can provide a more accurate assessment of bridge deck conditon than GPR alone (Maser 2009).

12.4.4 Acoutic Emission

An innovative and developing NDE technology for condition assessment of bridges is the development of AE for concrete. Traditionally used for crack monitoring in steel bridges, recent research has included several studies of AE activity during the cracking of concrete (Hsiung et al. 2000; Yoon et al. 2000; Colombo et al. 2003; Columbo et al. 2005; Prasad and Sagar 2008). Most studies have focused on laboratory testing of samples under controlled loading, and these studies have illustrated the increased AE activity associated with crack initiation and growth in concrete structures, as well as crack fretting, that is, acoustic noise generated from the rubbing of the crack face during opening and closing of a crack in response to applied load (Ono 2006; Gostautus and Tamutus 2007).

Consequently, analysis of the AE activity in a concrete structure during the controlled application of loads may provide a measure of the damage present in the structure, based on the level of AE activity present. The basic concept has been studied and captured using various statistical parameters to describe the AE response of a concrete structure to load (Colombo et al. 2003; Columbo et al. 2005; Ono 2006; Prasad and Sagar 2008). The results of this research could be applied through a load testing regime to assess concrete bridges such as adjacent box girder bridges, where important areas of the bridge members are inaccessible for visual assessment.

Using a load testing scheme could have the potential to allow the identification of concrete bridges that have significant in situ damage. Using AE technology, cracking in hidden areas can be assessed during a load test by placing sensor on the soffit of the bridge structure and detecting AE activity that initiates in the web area and propagates through the concrete. Higher levels of AE for bridges with significant in situ damage relative to bridge with little or no in situ damage could be used to identify and prioritize bridges for more in-depth inspections or further assessment.

Figure 12.16 shows AE results from a controlled load test of a box girder removed from the Lakeview Drive Bridge in Pennsylvania, which suffered a collapse of a soffit girder in December, 2005 (Naito et al. 2010). Subsequent to that collapse, load testing to failure was conducted on two members of the bridge for research purposes. The load testing consisted of a number of increasing load cycles leading up to the failure load for the member, and AE activity was monitored during the testing (Gostautus and Tamutus 2007). The onset of new damage to the concrete girders was assessed using AE statistical parameters. This research indicated that quantification of the AE results using these parameters was promising for tracking the damage in the beam during the testing, and it correlated well with damage in girders observed before testing (Gostautus and Tamutus 2007). Results suggested that a 90-ft. girder could be monitored from a single sensor location.

These results, combined with the other research previously conducted on using AE to assess the in situ condition of concrete structures, indicate that this innovative technology may have practical applications. For example, the technology may have application as a means of prioritizing bridge for more in-depth evaluation, by separating those structures with significant in situ damage from those with little or no in situ damage, through a simple load test. Such information can be used in reliability-based inspection decisions to identify bridges where inspection needs are greatest.

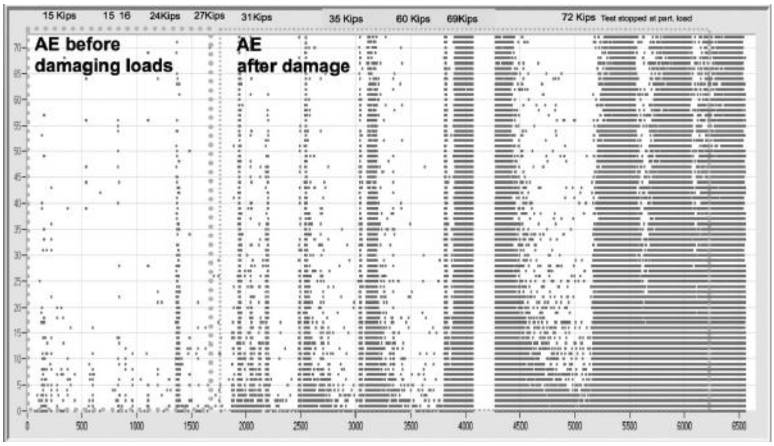


FIGURE 12.16 Acoutic emission (AE) results during load testing of prestressed box girders. (From Gostautus, R. S., and T. Tamutus, *Condition Assessment of Prestressed Concrete Girders from the Lakeview Drive (I-70) Bridge Using Acoustic Emission*, Western Pennsylvania Transportation Research Forum, Pittsburgh, PA, 2007.)

12.4.5 Radiographic Testing

Radiographic Testing is a NDT technology that is normally used during the fabrication of steel bridges to detect volumetric flaws in welds. Although seldom used for the condition assessment of in-service bridges, the technology has several advantages over other NDE technologies that make it an ideal solution for certain circumstances. First, the technology can assess inaccessible details and components where no other means of condition assessment are possible. Second, the results produced in the form of a radiograph show the shape, size, and location of the features in ~1:1 ratio with their actual size. As a result, interpretation of the results is relatively straightforward and can be easily understood by engineers. Third, the quality of the images produced can be assured through image quality indicators (IQIs) that appear on the radiograph, thereby reducing uncertainty in the success of the test. IQIs are controls in the form of wires, shims, or steps with holes of predetermined size that are included in the radiograph to determine the sensitivity of the image recorded (Cartz 1995; Saleh 2004).

Industrial radiography is generally implemented using either X-ray or gamma ray sources. X-ray sources are typically X-ray tubes or linear accelerators, which generate radiation through an electronic process of excited electrons colliding with a suitable target, which emits X-rays. In contrast, gamma rays are generated by decomposing isotopes, such as iridium or cobalt, and generally have lower energies, that is, less penetration capabilities than X-ray sources. Table 12.1 shows estimated penetration depths for gamma and X-ray sources, illustrating the generally increased penetration capabilities of the X-ray sources (Cartz 1995; Hellier 2001; IAEA 2002). Two of the most commonly used isotopes in concrete inspection are Cobalt-60 and Iridium-192, which can penetrate up to 20 in. of concrete (Cartz 1995; IAEA 2002). X-ray sources are capable of much greater penetration; they are typically used for concrete specimens of greater thickness than could be reliably examined using gamma rays, as shown in Table 12.1 (Bungey et al. 2006).

Radiography is based on the principle that as radiation is transmitted through a material, intensity of the transmitted radiation is lost due to absorption and scattering. Voids or areas of lower density within the material allow more radiation to pass through than does the surrounding material; this difference in the intensity of the radiation is then recorded on a film or detector (Cartz 1995; IAEA 2002). The contrast resolution of the radiographic image is typically 2%, although often 1% resolution can be obtained. As a result, the defect or feature to be detected in the radiographic image must represent at least a 1% change in density to be resolved on the film or X-ray image.

TABLE 12.1 Penetration Depths for Radiography

Source	Radiation Source/ Energy	Thickness Ranges in Steel (in.)	Thickness Ranges in Concrete (in.)
Gamma ray	Ir_{192}	0.5–2.4	1–10
	Co_{60}	1–5	5–20
X-ray	1 MeV	0.8–4.0	~3–14
	6 MeV	3–16	~11–50
	18 MeV	12–30	~ 20–63

Source: Data from Cartz, L., *Nondestructive Testing*, ASM International, Materials Park, OH, 1995; IAEA, *Guidebook on Non-Destructive Testing of Concrete Structures*, International Atomic Energy Agency, Vienna, Austria, 2002.

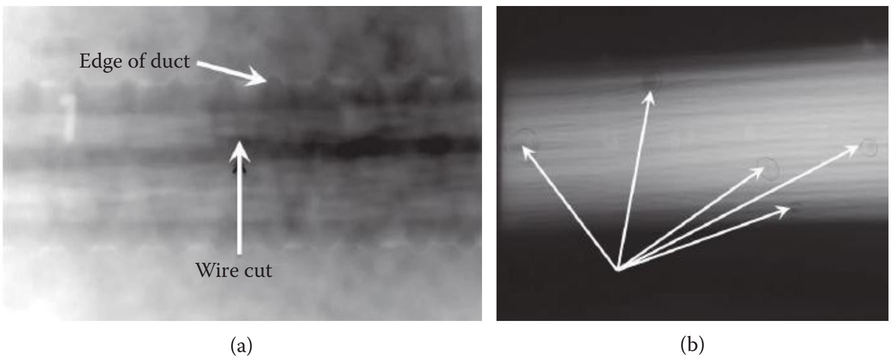


FIGURE 12.17 Radiograph scans for (a) cut wire inside grouted PT duct and (b) cut strand and wires inside cable stay mock-up.

Radiography for concrete bridges can detect and locate reinforcing bars, voids, cracks, and fractured prestressing strands. The technology can also be used to inspect swage fittings, cable terminations, suspension cables, and other inaccessible areas in bridges that require assessment. Research was conducted by the FHWA in 2004 that demonstrated the utility of this technology. Two examples are shown in Figure 12.17. Figure 12.17a shows a wire cut in a grouted PT tendon embedded in a 12-in.-thick concrete prestressed prism specimen. As shown in the figure, a single wire cut in the 7 wire, 0.5-in. diameter strand can be imaged through 12-in. concrete and grout. Figure 12.17b shows a cable-stay laboratory mock-up of the C&D canal bridge, Delaware. The laboratory mock-up shown in the image consisted of a steel pipe (3/8 in. wall), 81 bundled prestressing strands, and grout. Embedded flaws consisted of single wire breaks, groups of wire breaks, and cut strands. The high-energy (6 MeV) X-ray image shows the flaws embedded in the specimen and illustrates the utility of the technology for imaging the internal condition of tendons and cables that are otherwise inaccessible.

An example use of radiography in the field for evaluating a concrete bridge involved the inspection of posttensioning cables in a decommissioned approach ramp at the Fort Lauderdale, Florida Airport, in April 2002 (Pinna 2002; Saleh et al. 2002). Several grout flaws were successfully detected and confirmed during the demolition, with some voids as small as 0.25 in. in height detected. Voids in posttensing ducts were also detected using high-energy X-ray during the construction of the Zakim Bridge in Boston, Massachusetts (El-Beik 2002). In this case, images of internal posttensioning ducts through 12 in. of concrete displayed grout voids in the duct, and results were consistent with physical sampling.

The primary advantage of radiography is its ability to assess areas that are otherwise inaccessible. There are a number of disadvantages; the equipment can be bulky and difficult to maneuver in the field, and access to two sides of a member is *always* required. The technology can be high-cost, especially when an X-ray source is required. Safety measures must be undertaken while in the field; this can increase costs and the time taken to perform this technique.

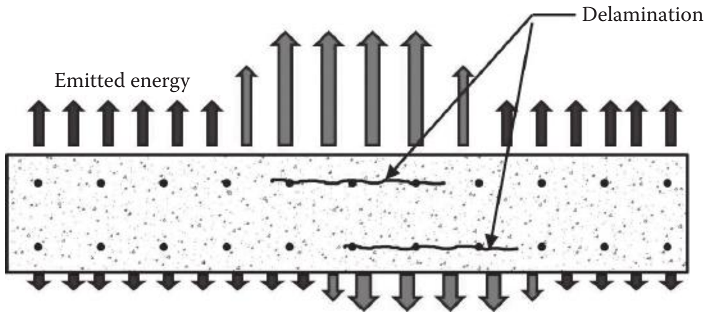


FIGURE 12.18 Schematic diagram of infrared radiation from the surface of a concrete deck.

12.4.6 Infrared Thermography

IR thermography as a tool for the condition assessment has been used in a limited fashion for a number of years. IR thermography is based on the principle that heat conduction through a material is affected by the presence of defects or discontinuities in the material and that this disruption of heat flow manifests in observable temperature variations at the surface of the material (Manning and Holt 1980; Manning and Masliwec 1993; Maser and Roddis 1990). These variations in surface temperature can be observed and recorded with IR cameras, which image the IR energy emitted from the surface. When a subsurface delamination exists in the concrete, it disrupts heat flow through the concrete. During the warming of the day, the area above the delamination warms more quickly than the intact concrete surrounding the delamination, resulting in increased IR energy being emitted from that area, as shown schematically in Figure 12.18. During a cooling phase overnight, the concrete surface above a subsurface delamination will likewise cool at a faster rate than the surrounding concrete and appear as a cooler area in an IR image (Washer 2009).

The equipment used in IR consists of a high-resolution IR camera, which records both the IR image and data relating to the image. Results are presented in real-time during the testing and can be stored for documentation purposes. Cameras are available commercially in formats hardened for field application.

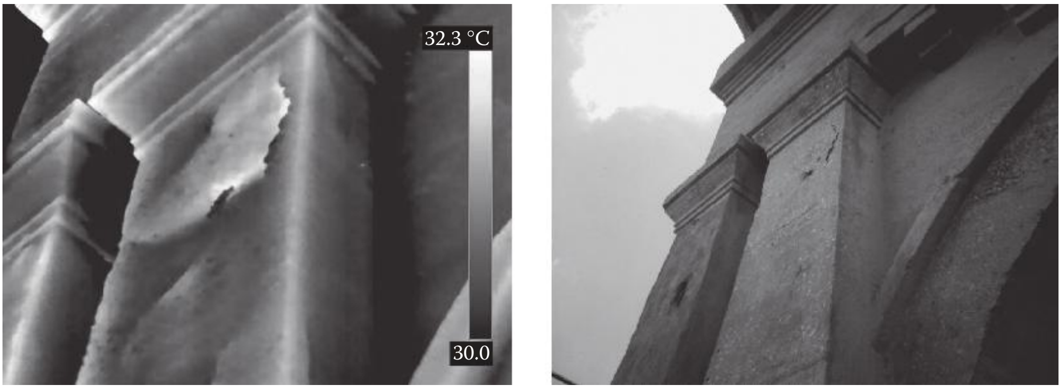
12.4.6.1 Applications

Specific guidelines describing the inspection conditions and timing for conducting thermal inspections of concrete bridge elements were developed under a pooled-fund study funded by the states of New York, Missouri, and Texas, and previous ASTM standards specifically for decks are also available (ASTM 2007; Washer 2009). New applications for this technology include the imaging of bridge soffits, where there is no solar loading (sunlight) available to warm the concrete. The soffit, while not directly exposed to sunlight, is still warmed throughout the course of the day by convective heating (Washer 2009), although the temperature contrast between delaminated areas and the surrounding material will not be as great as that seen in areas warmed by direct sunlight.

An example application of thermography for areas not exposed to direct sunlight (areas) is shown in Figure 12.18, which shows delaminations resulting from corrosion damage in the substructure of a historic bridge (Figure 12.19). Images such as these can be obtained from significant distances from the member being imaged, such that the technology has reduced access requirements relative of other NDE technologies including GPR, UPV, or IE. As a result, it is more practically implementable within a bridge inspection program as part of routine inspection tasks, to provide an additional tool for inspectors to assess damage and deterioration in the field.

12.4.6.2 Advantages and Disadvantages

IR thermography can be performed using handheld equipment and at a distance from the surface being tested and can therefore be easily performed without disrupting traffic flow or accessing the bridge superstructure. It is an area testing technique, rather than a point- or line-testing technique, such that it can assess



(a) (b)

FIGURE 12.19 Substructure damage of a historic bridge: (a) thermal images of delaminations and (b) standard photograph of the same area.

a sizeable area quickly. The results are immediate—the technology produces a thermal image without the need for further data processing; results can typically be confirmed on-site using hammer sounding or coring, if needed.

A significant disadvantage of this technique is that it depends on certain ambient environmental conditions to produce sufficient temperature gradients in the concrete to make subsurface defects to appear in the IR image (Washer 2009; Washer et al. 2009a and 2009b). For surfaces exposed to direct sunlight, long, uninterrupted periods of sun are desirable. For shaded surface, such as the bridge soffits, significant ambient temperature changes of at least 15°F are needed to produce the necessary conditions.

12.4.7 Magnetic Flux Leakage

MFL is an NDE technology with the potential for detecting fractured prestressing strands embedded in concrete and has been a topic of research for several years (Ciolko and Tabatabai 1999; Ghorbanpoor et al. 2000). Fracture of prestressing stands due to corrosion damage has led to beam collapse in the past, as previously discussed in Section 12.4.4.

The MFL method as shown in Figure 12.20 works by inducing a magnetic field within the prestressing steel strands and detecting the leakage of that field that results from sudden discontinuities in the strand (i.e., fractured strand or section loss) (Makar and Desnoyers 2001). The process of damage detection in the strand is analogous to the process involved in MT. For MT, finely divided iron particles are attracted to magnetic fields leaking from a crack in the surface of the steel. For MFL, the leaking magnetic fields are detected using coils, Hall effect, or superconducting quantum interference device sensors (Ghorbanpoor 1998; Krause et al. 2002). This enables detection of the leaking field through significant air gaps (or concrete cover) of between 5 in. up to more than 11 in., according to research results reported in the literature (Ghorbanpoor et al. 2000; Scheel and Hillemeier 2003). The method is shown schematically in Figure 12.20. Rare-earth magnets are typically used to provide opposing poles, separated by some distance such that the magnetic field between the poles penetrates the concrete and induces a magnetization in the embedded steel strand and any other ferromagnetic material (i.e., steel) in the area, such as stirrups or other mild reinforcing.

A sensor is used to measure the ambient magnetic field level at its position between the magnetic poles. The sensors and magnets form a sensor head that is scanned across the surface of the concrete axially aligned with the embedded steel strand. Sudden changes in the geometry of the embedded steel, such as a broken wire, result in a sudden change in the ambient magnetic field as the sensor head is scanned along the surface of the concrete (Gaydecki et al. 2007). Changes in the cross-sectional area of the steel within the aperture of the sensor head also results in variations in the ambient magnetic field

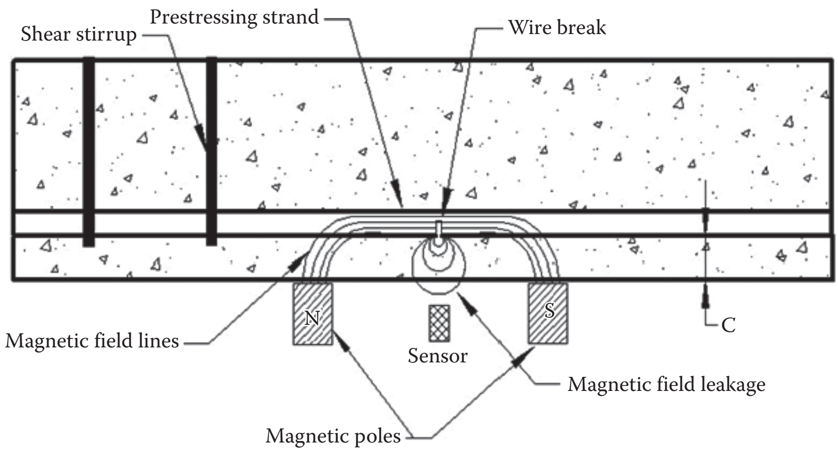


FIGURE 12.20 Magnetic flux leakage test schematic.

levels. These changes are less localized in nature relative to the response created by a fractured strand. Mild steel, such as stirrups and other reinforcing, also result in variations of the ambient magnetic field measured, which can complicate interpretation of results (Gaydecki et al. 2007). Varying concrete cover can also create variations in the measured ambient field. However, even with these recognized limitations, the MFL approach provides a potential solution to nondestructively detecting broken and corroding strands embedded in concrete.

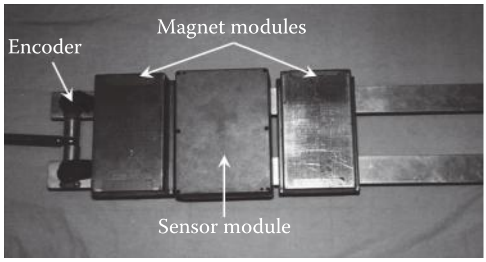
An example of the current state of the technology is shown in Figure 12.21. Figure 12.21a shows a plan view of an MFL unit; Figure 12.21b shows the orientation of an MFL unit in use on the soffit of a box girder bridge. This figure shows a MFL unit developed at the University of Wisconsin (Ghorbanpoor et al. 2000).

Research in the United States has typically focused on measuring the leakage field resulting from direct induction, that is, during magnetization. An alternative approach is to utilize remnant or residual magnetization resulting from magnetizing the embedded steel. Electromagnetics are used to magnetize the embedded steel from distance up to ~12 in. (Scheel and Hillemeier 2003). The resulting magnetization of the embedded steel, which remains (at a reduced level) after the electromagnet is removed, creates a dipole in the area of a fracture of the strand or wire (Makar and Desnoyers 2001; Scheel and Hillemeier 2003). Some research has suggested this method is more effective than induced magnetic fields; however, comparison data is limited.

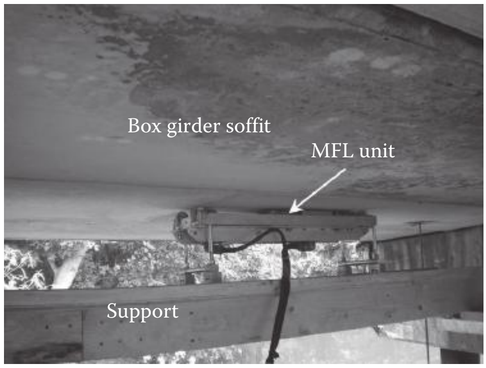
12.5 Future Directions in Bridge Inspection

Recent developments in the bridge inspection area include the development of guidelines for reliability-based inspections (RBI). This technology has the goal of improving the safety and reliability of bridges by optimizing inspection resources, focusing resources on the bridges most in need of inspection to ensure safety and reliability. Current requirements for bridge inspections stipulate fixed time intervals, typically 24 months, between inspection, regardless of the condition or durability of a particular bridge. Such a uniform time interval does not adjust to the particular inspection needs of a bridge; a new bridge in good condition is inspected at the same required interval as an aging structure that may be in much worse condition. In contrast, an RBI process characterizes the reliability of a particular bridge, based on a risk analysis, and applying an appropriate inspection interval matched to the needs for the bridge.

Ongoing research has developed draft guidelines for implementing this approach for inspection planning. Generally, the intention of the methodology is to identify those bridges that have good durability and reliability characteristics, such that the likelihood of damage developing is low, and inspection needs are less, and those bridges where the likelihood of damage developing is high, and therefore inspection



(a)



(b)

FIGURE 12.21 Magentic flux linkage (MFL) inspection unit. (a) MFL unit showing magnet modules, sensor modules, and distance encoder. (b) MFL system deployed in the field on the soffit of a box girder; support for MFL unit is also ((a) Image courtesy of Ghorbanpoor, University of Wisconsin.)

needs are greater. The methodology is based on well-established approaches in other industries that include risk-based assessments in the planning and execution of inspections. Under the RBI approach, the likelihood of certain damage occurring on a particular bridge and the consequences of that damage are assessed. The likelihood of damage occurring is assessed through an expert elicitation that considers the bridge design characteristics, environmental and structural loading on the bridge, and its current condition. Information on the deterioration patterns within a given bridge inventory can be used to support the assessment. These data are utilized in developing engineering rationale for determining the likelihood of damage occurring into one of four categories, ranging from "remote" to "high." The consequences of that damage occurring are also assessed through an expert elicitation, considering the criticality of different damage modes occurring. This process considers whether a given damage mode occurring will result in a benign impact on safety and serviceability or will have more significant impact. The consequence of the damage occurring is also categorized into one of only four categories, ranging from "low" to "severe." These data are then used to determine an appropriate inspection interval for a given bridge, by simply locating the various damage modes that are likely for a bridge within a risk matrix as shown in Figure 12.22. Bridges where damage is likely to occur, and/or consequences would be high if the damage did occur, tend to the upper right corner of the matrix. This indicates that shorter inspection intervals and more intense inspections would be appropriate. Bridges where damage is unlikely, or the consequence of that damage are small, tend toward the lower left corner, indicating longer inspection intervals and/or less intense inspections would be appropriate. Typical inspection intervals of 12–96 months are envisioned under such an RBI approach.

Data from the assessment are also used to identify inspection needs for a given bridge, based on the likelihood of the damage occurring and its consequence. This approach prioritizes bridge inspection needs. For example, fatigue cracking is more likely to occur in category E details in steel bridges, and therefore these details may require 100% hands-on inspection. Under an RBI approach, the consequences

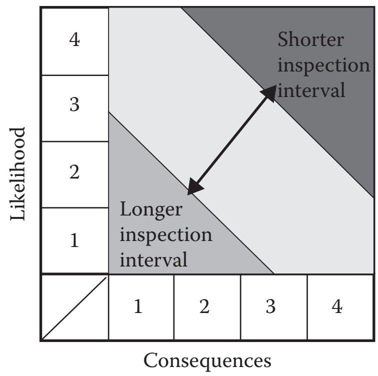


FIGURE 12.22 Risk matrix used to determine inspection interval.

of the fatigue crack developing is also assessed to determine if the fatigue crack would actually have significant consequences in terms of bridge safety or serviceability. If the bridge is multigirder, and the girders are closely spaced, the consequences of the fatigue crack may be very insignificant. As such, the detail may require less focus than a detail in a bridge that has heavy traffic volumes or less redundancy. The risks to the inspector resulting from difficult access challenges while conducting the inspection may not be justified if the cracking is unlikely to happen or of little consequence if it were to occur. On the other hand, if the cracking is likely or the consequences severe, the assessment may show that more intense inspections and/or NDE may be needed. The process improves the reliability of the inspection overall by focusing resources where most needed, as determined through an engineering assessment, as opposed to uniform inspection requirements that do not recognize variable risks associated with different bridges based on their design, materials, condition, and service levels.

Among the benefits of RBI is improved allocation of resources by focusing inspection efforts where most needed and reducing inspection requirements where inspections are adding little value. The quality of inspections can also be improved through the prioritization process, providing further guidance to inspectors to ensure critical damage modes are assessed in the field more uniformly. The concepts underlying the RBI process can be applied to bridges for the purpose of modifying inspection intervals, or to simply prioritize inspection efforts based on rational process, even if the interval between inspections is not changed. The RBI approach to inspection planning and execution is in the research stages.

References

AASHTO. 1997. *Commonly Recognized (CoRe) Structural Elements*. American Association of State Highway and Transportation Officials, Washington, DC.

Alampalli, S. and F. Jalinoos. 2009. "Bridge NDT: Use of NDT Technologies in US Bridge Inspection Practice." *Materials Evaluation,* 67(11), 1237–1246.

Alleyne, D. N., B. Pavlakovic et al. 2004. Rapid, long range inspection of chemical plant pipework using guided waves. *Proceedings of the 11th Asian Pacific Conference on Nondestructive Testing*, November 3, 2003 to November 7, 2003, Jeju Island, Republic of Korea, Trans Tech Publications Ltd.

ASTM. 2007. *Standard Test Method for Detecting Delaminations in Bridge Decks Using Infrared Thermography. Standard ASTM D4788 - 03(2007)*. ASTM International, West Conshohocken, PA.

ASTM. 2008. *Standard Guide for Magnetic Particle Testing. E709-08*. ASTM International, West Conshohocken, PA.

ASTM. 2009. *Standard Test Method for Pulse Velocity through Concrete*. ASTM International, West Conshohocken, PA.

ASTM. 2011. *Standard Practice for Liquid Penetrant Testing. E1417-11*. ASTM International, West Conshohocken, PA.

AWS. 2010. *Bridge Welding Code AWS D1.5*. American Welding Society, Miami, FL.

Barnes, C. L. and J.-F. Trottier. 2004. "Effectiveness of Ground Penetrating Radar in Predicting Deck Repair Quantities." *Journal of Infrastructure Systems,* 10(2), 69–76.

Bungey, J. H., S. G. Millard et al. 2006. *Testing of Concrete in Structures*. Taylor & Francis, New York.

Buyukozturk, O. 1998. "Imaging of Concrete Structures." *NDT&E International,* 31(4), 233–243.

Cardimona, S., B. Willeford et al. 2001. *Bridge Deck Condition Studies in Missouri Utilizing Ground Penetrating Radar*. Missouri Department of Transportation, Jefferson City, MO.

Cartz, L. 1995. *Nondestructive Testing*. ASM International. Materials Park, OH.

Cella, F. A. 1994. *Common Misapplications of NDT in the Field Inspection of Steel Bridges. Structural Materials Technology: An NDT Conference*, Atlantic City, NJ, Technomic Publishing Company, Inc.

Chiclott, A. and D. Roach. 2010. Comparative Vacuum Monitoring: A New Way to Monitor Cracks in Stuctures. *Structural Materials Technology 2010*. S. Alampalli and Jalinoos, F., editors. American Society for Nondestructive Testing, New York, 114–117.

Ciolko, A. T. and H. Tabatabai. 1999. *Nondestructive Methods for Condition Evaluation of Prestressing Steel Strands in Concrete Bridges. NCHRP Web Document*. Transportation Research Board, Washington, DC.

Clausen, J. S. and Knudsen, A. 2008. *Nondestructive Testing of Bridge Decks and Tunnel Linings Using Impulse-Response*. Ramboll Denmark, Bredevej, Denmark.

Clemena, G. G., M. G. Lozev et al. 1995. *Interim Report: Acoustic Emission Monitoring of Steel Bridge Members*. Virginia Department of Transportation, Richmond, VA.

Colombo, I. S., I. G. Main et al. 2003. "Assessing Damage of Reinforced Concrete Beam Using "B-Value" Analysis of Acoustic Emission Signals." *Journal of Materials in Civil Engineering,* 15(3), 280–286.

Columbo, S., M. Forde et al. 2005. "Predicting the Ultimate Bending Capacity of Concrete Beams from the 'Relaxation Ratio' Analysis of AE Signals." *Construction and Building Materials,* 19, 746–754.

Daigle, M., D. Fratta et al. 2005. *Ultrasonic and X-Ray Tomographic Imaging of Highly Contrasting Inclusions in Concrete Specimens*. American Society of Civil Engineers, Austin, TX.

Damgaard, N., S. Walbridge et al. 2010. "Corrosion Protection and Assessment of Weathering Steel Highway Structures." *Journal of Constructional Steel Research*, 66(10), 1174–1185.

Davis, A. G. 1994. *Novel Nondestructive Techniques for the Evaluation of Large Concrete Structures. Structural Materials Technology: An NDT Conference*, Technomic Publishing Company, Inc., Atlantic City, NJ.

Davis, A. G. and C. G. Peterson. 2003. *Nondestructive Evaluation of Prestressed Concrete Bridges Using Impulse Response. Nondestructive Testing in Civil Engineering 2003*. DGZIP, Berlin, Germany.

Delahaza, A., C. G. Petersen et al. 2008. Three Dimensional Imaging of Concrete Structures Using Ultrasonic Shear Waves. *Advanced NDE of Concrete Structures Workshop*. F. Jalinoos and G. A. Washer, editors. Federal Highway Administration, Oakland, CA.

Dilek, U. 2007. "Ultrasonic Pulse Velocity in Nondestructive Evaluation of Low Quality and Damaged Concrete and Masonry Construction." *Journal of Performance of Constructed Facilities*, 21(5), 337–344.

El-Beik, A. 2002. *X-Ray Examination of Post-Tensioning Ducts in Segmental Bridges on the CA/T Project*. Jack Moore Associates, Worcester, MA.

Everett, T. D. 2011. Personnel Communication/T-18 FHWA presentation. G. Washer. Columbia, MO.

FHWA. 1988. *Revisions to the National Bridge Inspection Standards (NBIS)*. Federal Highway Administration, Washington, DC.

FHWA. 1995. *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*. Federal Highway Administration, Washington, DC.

FHWA. 2004. Title 23: Highways. Section 650, Subpart C: National Bridge Inspection Standards, Federal Register. Federal Highway Administration, Washington, DC.

FHWA. 2010. *Technical Advisory 5140.31 "Inspection of Gusset Plates Using Nondestructive Evaluation Technologies."* Federal Highway Administration, Washington, DC.

Fowler, K. A., K. A. Elfbaum et al. 1997. "Theory and Application of Precision Ultrasonic Thickness Gaging." *Panametrics Applications Notes,* 2(10), 11. Waltham, MA. (http://www.ndt.net/article/wt1097/panam/panam.htm)

Friese, M., C. Roder et al. 2010. *Quality Control for Crack Repair—A New Approach Using Ultrasonic Imaging. NDE/NDT Technologies for Highways and Bridges: Structural Materials Technology (SMT)* 2010, Alampalli and Jalinoos, F., editors. American Society for Nondestructive Testing, New York, 374–382.

Gaydecki, P., B. Fernandes et al. 2007. "Inductive and Magnetic Field Inspection Systems for Rebar Visualization and Corrosion Estimation in Reinforced and Pre-Stressed Concrete." *Nondestructive Testing & Evaluation*, 22(4), 255–298.

Ghorbanpoor, A. 1998. Magnetic based NDE of Steel in Prestressed and Post-Tensioned Concrete Bridges. *Structural Materials Technology III: An NDT Conference*. R. Medlock and S. Chase, editors. San Antonio, TX, SPIE 22(4), 255–298.

Ghorbanpoor, A. and N. Benish. 2003. *Non-Destructive Testing of Wisconsin Highway Bridges*. Wisconsin Department of Transportation, Madison, WI.

Ghorbanpoor, A., R. Borchelt et al. 2000. *Magentic-Based NDE of Prestressed and Post-Tensioned Concrete Members—The MFL System*. Federal Highway Administration, Washington, DC.

Gostautus, R. S. and T. Tamutus. 2007. *Condition Assessment of Prestressed Concrete Girders from The Lakeview Drive (I-70) Bridge Using Acoustic Emission*. Western Pennsylvania Transportation Research Forum, Pittsburgh, PA.

Graybeal, B. A., R. A. Walther et al. 2000. Ultrasonic Inspection of Bridge Hanger Pins. *Structural Materials Technology V: An SMT Conference*. S. Alampalli, editor. Technomic Publishing Company, Inc., Atlantic City, NJ, 261–266.

Gucunski, N. and S. Nazarian. 2010. *Material Characterization and Condition Assessment of Reinforced Concrete Bridge Decks by Complementary NDE Technologies*. American Society of Civil Engineers, Orlando, FL.

Gucunski, N., F. Romero et al. 2010. "Multiple Complementary Nondestructive Evaluation Technologies for Condition Assessment of Concrete Bridge Decks." *Transportation Research Record: Journal of the Transportation Research Board,* Bridge Engineering 2010, 2, 34–44.

Gucunski, N., Z. Wang et al. 2008. "Automated Interpretation of Bridge Deck Impact Echo Data Using Three-Dimensional Visualization." NDE/NDT for Highways and Bridges: Structural Materials Technology (SMT). The American Society for Nondestructive Testing, Oakland, CA. 33–42.

Hellier, C. 2001. *Handbook of Nondestructive Evaluation*. McGraw-Hill Professional Publishing, Blacklick, OH.

Hsiung, S. M., A. Ghosh et al. 2000. "Fractal Pattern Characterization of Acoustic Emissions for Continuous Damage Assessment of Reinforced Concrete Structures." *Condition Monitoring of Materials and Structures,* Ansari, F. ed. American Society of Civil Engineers, Reston, VA, 2–20.

Hutton, P. H. and J. R. Skorpik. 1978. *Acoustic Emission Methods for Flaw Detection in Steel Highway Bridges*. Federal Highway Administration, Washington, DC, 58.

IAEA. 2002. *Guidebook on Non-Destructive Testing of Concrete Structures*. International Atomic Energy Agency, Vienna, Austria.

Januszkiewicz, M. S. 2010. *Bridge Hanger Pin Inspection Using Ultrasound. NDE/NDT Technologies for Highways and Bridges: Structural Materials Technology (SMT)*. American Society for Nondestructive Testing, New York, 265–272.

Jensen, P. and M. B. Johnson. 2010. *AASHTO Bridge Element Inspection Manual*, American Association of State Highway and Transportation Officials, Washington, DC.

Kee, S. H., T. Oh et al. 2011. "Nondestructive Bridge Deck Testing with Air-Coupled Impact-Echo and Infrared Thermography." *Journal of Bridge Engineering*, 17 (SPECIAL ISSUE: Nondestructive Evaluation and Testing for Bridge Inspection and Evaluation), 928–939.

Kinzel, A., C. Burgess et al. 1929. "Non-Destructive Testing of Welds by Means of the Stethoscope and X-ray." *Journal of the American Welding Society*, September, 71–78.

Kohl, C., M. Krause et al. 2003. "3D-Visualisation of NDT Data Using a Data Fusion Technique." *Insight: Non-Destructive Testing and Condition Monitoring*, 45(12), 800–804.

Kosnik, D. E., T. Hopwood et al. 2010. *Acoustic Emission Monitoring for Assessment of Steel Bridge Details. Review of Progress in Quantitative Non-Destructive Evaluation*. American Institute of Physics, San Diego, CA.

Krause, H. J., W. Wolf et al. 2002. "SQUID Array for Magnetic Inspection of Prestressed Concrete Bridges." *Physica C: Superconductivity*, 368(1–4), 91–95.

Krause, M., F. Mielentz et al. 2001. "Ultrasonic Imaging of Concrete Members Using an Array System." *NDT&E International*, 34(6), 403–408.

Krause, M., H. Wiggenhauser et al. 2002. NDE of a Post Tensioned Confete Bridge Girder Using Ultrasonic Pulse Echo and Impact Echo. *Structural Materials Technology V: An NDT Conference*. S. Alampalli and G. A. Washer, editors. Cincinnati, OH, American Society for Nondestructive Testing, New York, NY, 386–395.

Lesniak, J. R., D. J. Bazile et al. 1997. "Coating Tolerant Thermography for the Detection of Cracks in Structures." *Materials Evaluation*, 55(9), 961–965.

Limaye, H. S. 2008. *Experience of Testing Post-Tensioned Bridges with Impact-Echo Technique. NDE/NDT for Highways and Bridges: Structural Materials Technology (SMT)*. Oakland, CA, American Society for Nondestructive Testing, New York, NY, 188–193.

Lozev, M. G. 2001. *Big Thompson River Bridge Phased-Array Ultrasonic Evaluation—Technology Demonstration and Field Trial*. Edison Welding Institute, Columbus, OH.

Lozev, M. G., B. Grimmett et al. 2002. Ultrasonic Phased-Array Inspection of Bridge Pins and Eybars. *Structural Materials Technology V: An NDE/NDT Conference*. S. Alampalli and G. Washer, editors. Buffalo, NY, American Society for Nondestructive Testing, New York, NY.

Lozev, M. G., G. A. Washer et al. 1997. *Investigation of Delayed Cracking in Pivot Steel Box Girders*. Virginia Transportation Research Council, Richmond, VA.

Maierhofer, C. 2003. "Nondestructive Evaluation of Concrete Infrastructure with Ground Penetrating Radar." *Journal of Materials in Civil Engineering*, 15(3), 287–297.

Maierhofer, C., M. Krause et al. 2004. "Complementary Application of Radar, Impact-Echo, and Ultrasonics for Testing Concrete Structures and Metallic Tendon Ducts." *Transportation Research Record: Journal of the Transportation Research Board*, 1892(1), 170–177.

Makar, J. and R. Desnoyers. 2001. "Magnetic Field Techniques for the Inspection of Steel under Concrete Cover." *NDT&E International*, 34(7), 445–456.

Manning, D. G., and F. B. Holt. 1980. "Detecting Delaminations in Concrete Bridge Decks." *Concrete International*, 2(11), 34–41.

Manning, D. G. and T. Masliwec. 1993. "Application of Radar and Thermograpby to Bridge Deck Condition Surveys." Bridge Management: Inspection, Maintenance, Assessment and Repair Papers Presented at the Second International Conference on Bridge Management, Held at the University of Surrey, Guildford, UK, April 18–21.

Martin, J., K. J. Broughton et al. 2001. "Ultrasonic Tomography of Grouted Duct Post-Tensioned Reinforced Concrete Bridge Beams." *NDT&E International*, 34(2), 107–113.

Maser, K. R. 2008. *Integration of Ground Penetrating Radar and Infrared Thermography for Bridge Deck Condition Evaluation. NDE/NDT for Highways and Bridges: Structural Materials Technology (SMT)*. Oakland, CA, American Society for Nondestructive Testing, New York, 67–74.

Maser, K. R. 2009. Integration of Ground Penetrating Radar and Infrared Thermography for Bridge Deck Condition Evaluation. *7th International Symposium on Nondestructive Testing in Civil Engineering*. Nantes, France.

Maser, K. R. and W. M. K. Roddis. 1990. "Principles of Thermography and Radar for Bridge Deck Assessment." *Journal of Transportation Engineering*, 116(5), 583–601.

Matzkanin, G. A. and H. T. Yoken. 2009. *Nondestructive Evaluation for Detecting Hidden Corrosion. AMMTIAC Quarterly Newsletter*. Alionscience, Washington, DC.

Miceli, M., M. Moshier et al. 2010. Fatigue Crack Growth Activity Determination during Inspections on Highway Bridges in Three States. *Stuctural Materials Technology 2010*. S. Alampalli and F. Jalinoos, editors. American Society for Nondestructive Testing, New York, 103–108.

Moore, M., B. Phares et al. 2001. *Reliability of Visual Inspection for Highway Bridges*. Federal Highway Administration, McLean, VA.

Moore, M., B. Phares et al. 2004. *Guidelines for Ultrasonic Inspection of Hanger Pins*. Federal Highway Administration, Washington, DC, 99.

Nair, A. and C. S. Cai. 2010. "Acoustic Emission Monitoring of Bridges: Review and Case Studies." *Engineering Structures*, 32(6), 1704–1714.

Naito, C., R. Sause et al. 2010. "Forensic Examination of a Noncomposite Adjacent Precast Prestressed Concrete Box Beam Bridge." *Journal of Bridge Engineering*, 15(4), 408–418.

NYSDOT. 1997. *Bridge Inspection Manual*. E. Andre Bigos, editor. New York State Department of Transportation, Albany, NY.

Ono, K. 2006. "AE Methodology for the Evaluation of Structural Integrity." *Advanced Materials Research*, 13–14, 17–22.

Parrillo, R., A. Haggan et al. 2006. Bridge Deck Condition Assessment using Ground Penetrating Radar. *9th European NDT Conference (ECNDT)*. Berlin, Germany.

Pinna, M. 2002. *Inspection of Post Tension Cables in concrete using High Energy X-rays, Ramp D at the Ft Lauderdale Airport*. HESCO, LA Honda, CA.

Pla-Rucki, G. F. and M. O. Eberhard. 1995. "Imaging of Reinforced Concrete: State-of-the-Art Review." *Journal of Infrastructure Systems*, 1(2), 134–141.

Prasad, B. K. R. and R. V. Sagar. 2008. "Relationship between AE Energy and Fracture Energy of Plain Concrete Beams: Experimental Study." *Journal of Materials in Civil Engineering*, 20(3), 212–220.

Prine, D. W. and D. R. Marron. 1997. *Acoustic Emission and Strain Gage Monitoring of Prototype Retrofit for Caltrans Structure B-22-26 R/L, I-80 Sacramento River (Bryte Bend) Sacremento, CA*. Infrastructure Technology Institute, Northwestern University, Evanston, IL.

Rens, K. L., D. J. Transue et al. 2000. "Acoustic Tomographic Imaging of Concrete Infrastructure." *Journal of Infrastructure Systems*, 6(1), 15–23.

Rose, J. L. 2002. "A Baseline and Vision of Ultrasonic Guided Wave Inspection Potential." *Journal of Pressure Vessel Technology*, 124(3), 273–282.

Roth, D. J., R. P. Tokars et al. 2010. "Ultrasonic Phased Array Testing for an Isogrid Structural Element with Cracks." *Materials Evaluation*, 68(1), 76–83.

Saleh, H. H. 2004. *Guidelines for Selection of X-Ray Radiography Systems for Evaluation of Highway Structures. Introduction to Nondestructive Evaluation Technologies for Bridges*. Federal Highway Administration, NDE Center, McLean, VA.

Saleh, H., J. J. Goni et al. 2002. Radiographic Inspection of Post-Tensioning Tendons Using High-Energy X-Ray Machine. *Structural Materials Technology V: An NDT Conference*. S. Alampalli and G. Washer, editors. Cincinnati, OH, American Society for Nondestructive Testing, New York, NY.

Sansalone, M. J. and N. J. Carino. 1998. "The Impact-Echo Method." *Concrete International,* 10(4), 38–46.

Scheel, H. and B. Hillemeier. 2003. "Location of Prestressing Steel Fractures in Concrete." *Materials in Civil Engineering*, 15(3), 7.

Schubert, F., H. Wiggenhauser et al. 2004. "On the Accuracy of Thickness Measurements in Impact-Echo Testing of Finite Concrete Specimens—Numerical and Experimental Results." *Ultrasonics*, 42(1–9), 897–901.

Scott, I. 1991. *Basic Acoustic Emission*. Gordon and Breach Science Publishers, New York.

Shenefelt, G. A. 1971. *Ultrasonic Testing Requirements of the AWS 1069 Building Code and Bridge Specifications.* Defense Technical Information Center, Fort Belvoir, VA, 342.

Shin, H. and D. Grivas. 2003. "How Accurate is Ground-Penetrating Radar for Bridge Deck Condition Assessment?" *Transportation Research Record: Journal of the Transportation Research Board*, 1845(1), 139–147.

Shokouhi, P., N. Gucunski et al. 2006. Using Impact Echo for Nondestructive Detection of Delamination in Concrete Bridge Decks. *NDE Conference on Civil Engineering*, *I*. al-Qadi and G. Washer, editors. St. Louis, MO, American Society for Nondestructive Testing, New York, NY, 285–290.

Streicher, D., D. Algernon et al. 2006. Development and Combined Application of NDT Echo-Methods for the Investigation of Post-Tensioned Concrete Bridges. *3rd International Conference on Bridge Maintenance, Safety and Management—Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost*, July 16, 2006 to July 19, 2006, Taylor & Francis/Balkema, Porto, Portugal.

Thomas, M. D. A., D. Weise et al. 1997. Three-Dimensional Visualization of Concrete Structures. *Innovations in Non-Destructuve Testing of Concrete*. S. Pessiki and L. Olson, editors. American Concrete Institute, Farmington Hills, MI, 233–243.

Tinkey, Y., P. K. Miller et al. 2008. *Internal Void Imaging Using Impact Echo Scanning. NDE/NDT for Highways and Bridges: Structural Materials Technology (SMT)*. New York, NY, American Society for Nondestructive Testing, New York, NY, 194–200.

Tinkey, Y. and L. D. Olson 2007. *Impact-Echo Scanning for Grout Void Detection in Post-Tensioned Bridge Ducts—Findings from a Research Project and a Case History*.Long Beach, American Society of Civil Engineers, Reston, VA.

Vannoy, D. W., M. Asmi et al. 1987. *Acoustic Emission Monitoring of the Woodrow Wilson Bridge*. Baltimore, MD, Maryland Department of Transportation, Hanover MD.

Warhaus, J. P., J. E. Mast et al. 1993. *Advanced Ground-Penetrating Imaging Radar for Bridge Inspection*. Lawrence Livermore National Laboratory, Livermore, CA.

Washer, G. and P. Fuchs. 2001. Phased Array UItrasonic Measurement in Bridge Pins. *FHWA NDE Validation Center Internal Reports*. Federal Highway Administration, McLean, VA.

Washer, G. A. 2009. *Application of Infrared Thermography (IR) for Detecting Defects in Concrete*. American Society for Nondestructive Testing, New York, NY.

Washer, G. A., R. G. Fenwick et al. 2009a. "Effects of Solar Loading and Other Environmental Conditions on Thermographic Imaging of Subsurface Defects in Concrete." *AIP Conference Proceedings*, 1096(1), 1489–1496.

Washer, G. A., R. G. Fenwick et al. 2009b. *Development of Handheld Thermographic Inspection Technologies*. Missouri Department of Transportatoin, Jefferson City, MO.

Washer, G. A., D. Lamtenzan et al. 2000. *Detection and Sizing of Cracks in Structural Steel Using the Eddy Current Method*. U.S. Department of Transportation, Washington, DC.

Wiggenhauser, H. and H. W. Reinhardt. 2010. "NDT in Civil Engineering: Experience and Results of the for 384 Research Group." *AIP Conference Proceedings*, 1211(1), 47–54.

Woodward, C. and A. McGarvie. 2000. Long Range Ultrasonic Fatigue Detection. *Structural Materials Technology IV: An NDT Conference*. S. Alampalli, editor. Technomic Publishing Co., Atlantic City, NJ, 267–271.

Wouters, J. and R. W. Poston. 2001. *Applications of Impact-Echo for Flaw Detection*. American Society of Civil Engineers, Reston, VA.

Yoon, D. J., W. J. Weiss et al. 2000. "Assessing Damage in Corroded Reinforced Concrete Using Acoustic Emission." *Journal of Engineering Mechanics*, 126(3), 273–283.

Zhu, J. and J. S. Popovics. 2006. Air-Coupled Impact-Echo Imaging of Concrete Structures. *NDE Conference on Civil Engineering*, St. Louis, MO. I. al-Qadi and G. A. Washer, editors. New York, American Society for Nondestructive Testing, New York, NY, 423–429.

Zilberstein, V., K. Walrath et al. 2003. "MWM Eddy-Current Arrays for Crack Initiation and Growth Monitoring." *International Journal of Fatigue*, 25(9–11), 1147–1155.

13

Bridge Inspection

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13.1 Introduction

13.1.1 Purpose

The purpose of the bridge inspection is to ensure safety of both the inspection team and the traveling public who use the bridges, and to ensure that the bridge structures provide continuous service to their users until repaired or replaced (Figure 13.1).

The U.S. Congress passed the Federal-Aid Highway Act of 1968, establishing the *National Bridge Inspection Standards* (NBIS). The NBIS sets the national requirements regarding bridge inspection procedure, inspection frequency, inspector qualifications, reporting format, and rating procedures, as defined in Title 23, Code of Federal Regulations, Part 650, Subpart C. The Federal Highway Administration (FHWA) *Inspector's Reference Manual* (FHWA 2012), the American Association of State Highway and Transportation Officials (AASHTO) *Manual for Bridge Evaluation* (AASHTO 2013), and the FHWA *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (FHWA 1995) along with the NBIS provide the requirements for bridge inspections. This chapter will discuss the basic fundamental requirements of bridge inspections.

13.1.2 Qualifications and Responsibilities of Bridge Inspectors

The program manager is, at a minimum, a licensed professional engineer and/or has 10 or more years of experience inspecting bridges in the field and who is current in FHWA-approved comprehensive bridge inspection training.

FIGURE 13.1 Ensure safety of all bridge components.

The team leader of the teams that perform the field inspections of the bridges has at a minimum a bachelor of science in engineering and has passed the Fundamentals of Engineering exam. Refer to the NBIS for further information and requirements including quality assurance and quality control.

13.1.3 Frequency of Bridge Inspection

Bridges are routinely inspected at regular intervals of 24 months or less. Some bridges are inspected more frequently to monitor rapidly changing deficient conditions, for example, where the load capacities are low, or where there are critical scour conditions. Underwater inspections of bridges, at regular intervals of 60 months or less, identify and quantify the extent of scour or deficient conditions below the water surface. Fracture critical inspections of structural steel details are at regular intervals of 24 month or less.

13.2 Pre-Inspection

13.2.1 Preparation for the Bridge Inspection

1. Gather and review information on previous conditions identified at the structures during previous inspections. Arrange for any special equipment needed to provide physical access.
2. Identify and find solutions to access issues, make appropriate preparations for confined or enclosed space during the inspections, and arrange for ventilation or breathing support if needed. Arrange for special rigging or equipment to provide access to components of trusses, long span structures, pin and hanger assemblies, truss eyebars and other components to allow visual inspection, and the use of additional inspection tools as appropriate (Figure 13.2).
3. Determine narrow shoulder widths or other operational concerns from aerial photos. Contact traffic control support to discuss and implement the traffic control plan.
4. Contact the railroad if necessary to schedule flaggers and time windows of closures to allow safe inspections of structures over or under railroads.

FIGURE 13.2 Gather pre-inspection information on bridge components and access.

5. Review load ratings and identify needs for field measurements and identifications of materials of the structure.
6. Determine types of foundations from as-built plans or previous bridge reports. Identify needs for information on conditions of the soil, water, and other forces interacting with the structure.
7. Identify the time of year when the channel is dry or the water is at its lowest. If there are controlled water flows in the channel, contact the water master about shutting down or slowing down flows in the channel during the inspection.
8. For steel structures, develop a fracture critical inspection plan that identifies fracture critical elements (members' fracture critical systems, weld details, and connections such as pin and hanger connections and eyebars) on as-built plans. Special emphasis should be placed on this inspection plan for structures whose superstructure has a lack of redundancy.

13.2.2 Tools of Inspection

Prior to departing on an inspection trip, consider the equipment that will be needed for a complete inspection of the types of bridges and sites visited during the inspection (FHWA-NHI 2012). The following list is a starting point for the types of gear necessary:

1. Safety: Cell phone, communication systems, diving communication systems, first aid kit, air monitor, gloves, climbing gloves, safety glasses, safety goggles (tinted for sun and clear for dark areas), ear plugs, ear muffs, hard hat, leather safety boots with waffle bottoms and safety toes, float jacket, knee pads, reflective jacket and pants, diving gear, harness and lanyard for climbing, climbing helmet, water and food to survive 3 days
2. Access: Waders, wading boots, float tube, air pump, flippers, small flat-bottomed boat, larger boat of appropriate size for open water/ocean (Figure 13.3), vehicle in reliable condition, lift trucks, under bridge inspection trucks with articulating arm, bolt cutters, extra locks, ropes of assorted lengths, wire brush
3. Measurement: Calipers, micrometer, angle meter to measure angle of high load hits, punch to mark ends of cracks, string line, straight edge with scale, measuring tapes of assorted sizes, channel cross section measuring tape with weighted end, folding rule, chains to chain deck to

FIGURE 13.3 Determine safe access in marine environments.

determine areas of delaminations, rod with scale for wade and probe inspections at substructures or foundations, geology pick, drill, dye penetrant, crack comparator to measure deck cracks

4. Field documentation: Camera with extra batteries and battery charger, clipboard, keel, chalk, spray paint to mark and measure deck deterioration
5. Bridge report and load calculations: Computer, monitor, keyboard, mouse, printer, software, database, calculator, reference books and manuals

13.3 Inspection

13.3.1 Access and Safety

Safe access is needed to examine the structure. Access issues may include locked gates or barriers with; deep, swift water in the channel; live traffic on or under the structure; live railroad tracks on or under the structure; electric rails and power lines; heights or confined spaces requiring special access techniques, equipment, or machinery; thick vegetation; encasements around structures; unsanitary homeless camps; and soot, debris, and snow removal for visibility.

At least two inspectors should be present at the bridge site for safety, for quality of data, and for training opportunities, and at least one of the inspectors needs to be a qualified team leader. Additional personnel are often needed to assist with entry into confined spaces, climbing, fracture critical inspections, underwater inspections, and traffic handling. A multidisciplinary team of professional civil, electrical, and mechanical engineers is needed to adequately inspect movable bridges.

13.3.2 Deck and Deck Joints

When asphalt concrete overlays cover the top of the deck, record the thicknesses of asphalt concrete overlays for use in determining load ratings. Record the locations, conditions, temperatures, and widths of joints, joint seals, and joint seal assemblies (Figure 13.4).

In reinforced concrete decks, record locations, orientations, lengths, widths, and spacings of deck cracks on the deck surfaces, including the deck soffit. Identify locations of deck spalling (the limits of

FIGURE 13.4 Bridge deck and joints.

which are usually identified audibly by dragging a deck chain over the deck surface) and record the extent of deck spalling. Use chaining and spray paint to determine locations and to record the extent of incipient spalls; the spalls and incipient spalls should be recorded on a map of the deck that has been drawn to scale. Map locations, areas, and depths of deck spalls. To model distributions of live loads on the deck to the superstructure girders or beams used in load ratings, note whether the deck material has an uncracked or a cracked section.

In steel decks, record locations of corrosion, section losses, deformations, and evidence of fatigue due to cyclic loading. Steel decks may be covered with a surface treatment. Look for locations where the surface treatment has started to fail to identify spots for a closer inspection, using traffic control if necessary or from beneath the deck. Steel decks should be inspected from the lower deck surface at the deck soffit with careful attention to locations of deformation, warping, or cracking. The inspector should inspect visually and identify areas that need closer attention, recording the location or each suspect area, so that a systematic and ongoing inspection and maintenance program can be established.

In timber decks, record locations and extent of decay or splitting, failure of connections, and missing or failed deck planks.

13.3.3 Approach Roadway

Record locations, orientations, lengths, widths, and spacing of cracks in approach slabs. Look for settlements of the embankments and determine causes of settlements.

Check for presence and appropriateness of warning signs for vertical or horizontal clearances, based on field measurements taken during the bridge inspection. Check the adequacy of the warning signs, object markers, and delineators for narrow or one-lane bridges and for posted bridges per the *Manual on Uniform Traffic Control Devices* (MUTCD), (FHWA 2009). Check presence, visibility, and appropriateness of load posting signs. Warning lights on movable bridges should be evaluated at the time of inspection to confirm operation and visibility (Figure 13.5).

FIGURE 13.5 Movable bridge with structural, mechanical, and electrical systems.

FIGURE 13.6 Bridge rails.

13.3.4 Rails and Approach Rails

Look along the tops of the bridge rails for sags and if sags are present, determine their causes (Figure 13.6).

Record locations and sizes of traffic impact damages and exposed reinforcing bars.

Look at the heights and levels of effectiveness of the bridge rails and approach roadway rails to channel errant traffic into the proper lane with positive smooth transitions between the approach rails and bridge rails. Confirm that approach guardrails have positive connections to the bridge rails. Check the adequacy of the approach rail end treatments.

13.3.5 Superstructure

Following the moment and shear diagrams for the bridge, identify and record causes and locations where distress can be seen due to moment, shear, and torsional shear. Descriptions of deformation, buckling, cracks, or areas of distress should include lengths, widths, orientation (longitudinal, transverse, diagonal, and patterned), and spacing.

For steel structures, follow the fracture critical inspection plan (FHWA-NHI 2011; Fisher 1984) developed during the pre-inspection that identifies fracture critical elements (members' fracture critical systems, details, and connections such as pin and hanger connections and eyebars) on as-built plans, and that describes safe access, procedures and frequency. Cracks can form at connections and joints due to stress concentrations (Figure 13.7). Visual and audible examinations of the superstructure may reveal cracks in steel accompanied by visible corrosion stains or produce loud noises under traffic as the two sides of the cracks clap open and close. However, crack identification and investigations should involve cleaning debris accumulation from the steel members, and hands-on testing to determine the points of crack initiation and the lengths of crack propagation. Where there are previous records available, the lengths of additional crack propagations or cracks that have formed in areas that have previously been repaired should be scrupulously documented and a follow-up repair plan developed. The inspector should record locations, orientations, lengths, and causes of each crack. Compare the current tip locations with dated punch marks at the previous tip locations and note rates of changes. Determine which cracks are in need of immediate repair. Immediate repairs of the cracks can be accomplished cost effectively by certified welders present on site during the inspection if there is access to the locations or post-inspection if there are numerous locations that require detailed repairs with related specialty inspection (AASHTO/AWS 2011). Elements prone to cyclic loading and resultant fatigue cracking or fracture as identified during the pre-inspection plan review should be examined in detail on a more frequent basis and documented. Identify signs of wear, fatigue cracks, or torsional strain cracks on steel pin and hanger assemblies, and if problems are identified, determine if temporary supports are needed

FIGURE 13.7 Fracture critical bridge superstructure.

and replace as soon as possible. For bridges identified as having primary truss gusset plates, record locations of buckling, describe the extent of corrosion and related section losses, measure and determine percentages of section losses for load ratings, and update load ratings to reflect changes in connections.

Steel structures must have a protective coating to prevent corrosion. Areas of imminent coating failure should be identified and an estimate of the remaining life of the coating should be included within the report. If spot repairs of the coating may cost effectively correct the areas where coating failure is imminent, quantities should be recorded, and all locations where work is necessary should be recorded.

In steel "I" members, identify locations of local buckling or warping of the compression flange, horizontal translation of the compression flange and web buckling, and compare to previous conditions. Note if the steel girders are composite or noncomposite with the deck for load rating, watch for slight movements between the deck and girders under traffic to identify girders that are behaving as noncomposite.

For high load hits, record widths, heights, and depths of spalls or deformations and exposed or damaged reinforcing bars or prestressing strands, and observe the patched spalls and estimate the number of previous high load hits. Estimate the load carrying capacity of the girder and determine if temporary shoring should be installed until permanent repairs can be completed. Traffic management may be necessary in the area above the damaged girders. Plan traffic control on or below the structure, if needed to protect temporary shoring and to provide positive protection for workers who will complete necessary repairs.

Record checks, splits, locations, and extents of decay and section losses in timber members for load ratings. Sound and drill to determine extents of V or interior rot inside timber members that are found at the bearing ends (WWPA 2011).

13.3.6 Joints and Hinges

Identify the structural functions of joints, for example, a hinge joint of a superstructure, or a joint at the end of a simple span. Identify the sources of problems with joints and hinges, which may be at the joint itself or as a result of failures of joint seals that were supposed to protect the joint from debris and moisture. Listen for noises of parts banging together and determine the source locations and causes. Failed joint seals can result in debris caught inside a joint that hinders expansion and causes distress in the superstructure. Failures of bearings below the joints may be visible with differentials in deck surface elevations on either side of the joint. Failed joint seals can result in water penetration at the ends of steel members below, producing corrosion.

Where steel members join, look for and record locations where rivets have broken off, where bolts have sheared off, and where cracks have appeared in tension zones of connections and where built-up members join, buckled plates. In pin and hanger joints, check that the nuts are intact at both ends of pins. Fracture critical members and joints are inspected hands-on at arm's length or closer, with care (Figure 13.8).

13.3.7 Substructure

Following the shear and moment diagrams, distress to the substructure can be caused by seismic forces, hydraulic forces including flows against debris piles, lateral spreading, horizontal earth movements, and vertical differential earth settlements (Figure 13.9).

Record locations and extents of section losses in steel members for load ratings. Record checks, splits, locations and extents of decay and section losses in timber members for load ratings. Sound and drill to determine extents of V or interior rot inside timber members, then plug the drilled hole with a wooden peg and seal.

13.3.8 Foundations

Measure and take photos of exposed foundations when the channel is dry or the water in the channel is low (Figure 13.10).

Compare the log of test borings with field conditions and note the rates of changes in the field conditions, including differential settlements of the structure.

FIGURE 13.8 Bridge joint.

FIGURE 13.9 Bridge substructure.

Follow the underwater inspection plan that identifies structural elements and describes safe access, procedure, and frequency to be used in the underwater inspection.

13.3.9 Scour and Channel

Before going into the channel, record the depths of water in the channel next to each of the abutments, bents, and piers and at the thalweg. Measure the channel cross section, with data included at abutments, bents, or piers, at thalweg, and at changes in slope at the bottom of the channel.

Record locations, quantity, and sizes of woody debris caught on bents or piers. Record sizes and locations of scour holes. Probe near the lower portions of the channel near the substructure or foundation to

FIGURE 13.10 Bridge foundation.

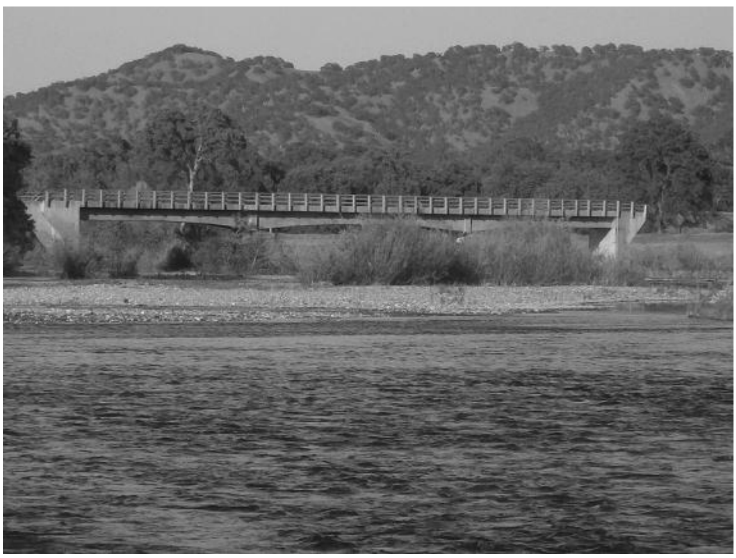

FIGURE 13.11 Scour at both ends of bridge.

determine if scour is present and if so, measure the extents of scour near exposed foundations, including spread footings and piles.

Identify scour critical bridges and develop a plan of action for these bridges, follow-up on critical findings, check for changes in conditions after flood or high flow events, and potential for structural instability in future events (Figure 13.11).

Look at the effectiveness of the scour system to protect the substructure and foundations not only under the current conditions but also under future event flow conditions. Look at the channel banks near the bridge for erosion that might be an indication of erodible soil and potential for scour at the bridge (Figure 13.12).

FIGURE 13.12 Eroded channel banks near the bridge.

For more information, refer to the FHWA-NHI publications: *Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18 (HEC-18)* (FHWA-NHI 2001), *Stream Stability and Scour At Highway Bridges* (FHWA-NHI 2005), *Plan of Action (POA) for Scour Critical Bridges*, (FHWA-NH 2007), and *Stream Instability, Bridge Scour, and Countermeasures: A Field Guide for Bridge Inspectors* (FHWA-NHI 2009).

For bridges located over navigational channels, measure and record the navigational clearances, both vertical and horizontal. Check the presence and conditions of the navigational warning systems. Check the condition and effectiveness of the dolphin systems that protect piers from boat impacts.

13.4 Post-Inspection

13.4.1 Bridge Report

A bridge report summarizes the condition, findings, and recommendations. There should be a consistent message throughout the bridge report, including updated load ratings, condition data, condition text, diagrams, photos, and work recommended.

Refer to Appendix B, page B-13 of the *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* (FHWA 1995) that shows an example of the minimum required data reported to the FHWA in the standardized format. In addition to the minimum, the bridge inspection report should describe the conditions and rates of changes in the conditions consistently throughout the data, the narrative commentary, the photos, and the load ratings. Sketches, channel cross sections, and clearance diagrams with current measurements taken in the field should be included as needed.

13.4.2 Recommended Work

Work recommendations in the bridge report usually have a general scope of work, a cost estimate, and the time when the work needs to be completed (Figure 13.13). Work recommendations might include treating a deck crack with methacrylate, removing unsound concrete and patching spalls, cleaning and painting steel members (Figure 13.14), repairing damage due to long-term fatigue of steel members, replacing deteriorated joint seals or joint seal assemblies, repairing or replacing deteriorated members and bearing assemblies, restoring load capacity, restoring the scour protection system, and adding or revising clearance signs (Figure 13.15). Early implementations of the recommended work can prevent exponential escalations in deterioration and costs.

FIGURE 13.13 Existing bridge and bridge replacement.

FIGURE 13.14 Bridge superstructure paint.

FIGURE 13.15 Animal crossing with clearance signs.

13.5 Summary

Bridge inspections per the NBIS protect the traveling public by preserving structural safety of the bridges. Preplanning is needed to understand the structure and to provide access while keeping the bridge inspectors safe. During the inspection, changes and rates of changes in conditions are identified, recorded, and the causes are interpreted. The data and findings, including recommended work, are gathered into the bridge report. The bridge data are reported to the FHWA, generating funding for repairs to keep our bridges safe.

References

AASHTO. 2013. *The Manual for Bridge Evaluation,* 2nd Edition, with 2011 and 2013 Interim Revisions, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO/AWS. 2011. *D1.5M/D1.5:2010, Bridge Welding Code*, 6th edition (with 2011 Interim Revisions), American Association of State Highway and Transportation Officials, Washington, DC.

FHWA. 1995. *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*, Report No. FHWA-PD-96-001, U.S. Department of Transportation, Federal Highway Administration, Washington, DC.

FHWA. 2004. *National Bridge Inspection Standards (NBIS)*, Code of Federal Regulations, Title 23, Part 650, Subpart C, U.S. Government Printing Office, Washington, DC.

FHWA. 2009. *Manual of Uniform Traffic Control Devices*, U.S. Department of Transportation, Federal Highway Administration, Washington, DC.

FHWA. 2012. *Bridge Inspector's Reference Manual (BIRM)*, FHWA-NHI 12-053, U.S. Department of Transportation, Federal Highway Administration, Washington, DC.

FHWA-NHI. 2001. *Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18 (HEC-18)*, FHWA Publication Number NHI-01-001, 4th edition, U.S. Department of Transportation, Federal Highway Administration, Washington, DC.

FHWA-NHI. 2005. *Stream Stability and Scour At Highway Bridges (Participant's Workbook)*, FHWA-NHI 05-019, U.S. Department of Transportation, Federal Highway Administration, Washington, DC.

FHWA-NHI. 2007. *Plan of Action (POA) for Scour Critical Bridges (CD)*, U.S. Department of Transportation, Federal Highway Administration, Washington, DC.

FHWA-NHI. 2009. *Stream Instability, Bridge Scour, and Countermeasures: A Field Guide for Bridge Inspectors*, FHWA-NHI 08-106, U.S. Department of Transportation, Federal Highway Administration, Washington, DC.

FHWA-NHI. 2011. *Fracture Critical Inspection Techniques for Steel Bridges*, FHWA-NHI 11-015, U.S. Department of Transportation, Federal Highway Administration, Washington, DC.

FHWA-NHI. 2012. *Safety Inspection of In-Service Bridges (Participant Workbook)*, Volumes 1 and 2, FHWA-NHI 12-042 and 12-043, U.S. Department of Transportation, Federal Highway Administration, Washington, DC.

Fisher, J. W. 1984. *Fatigue and Fracture in Steel Bridges: Case Studies*, Wiley-Interscience, New York, NY.

WWPA. 2011. *Western Lumber Grade Rule Book*, Western Wood Products Association, Portland, OR.

Relevant Websites

FHWA. 2004, revised 2009. National Bridge Inspection Standards: http://www.fhwa.dot.gov/bridge/nbis.htm.

FHWA. 2012. *Bridge Inspector's Reference Manual*: http://www.fhwa.dot.gov/bridge/bripub.htm.

FHWA. 2007. Plan of Action for Scour Critical Bridges: http://www.fhwa.dot.gov/engineering/hydraulics/bridgehyd/poaform.doc.

FHWA-NHI. 2001. *Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18*: http://isddc.dot.gov/OLPFiles/FHWA/010590.pdf.

14

Steel Bridge Evaluation and Rating

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14.1 Introduction

Evaluation of existing bridges has become increasingly important with the aging and deterioration of structural components as well as the growing demands for traffic volume and weight. Many bridges have accumulated a huge amount of loading cycles during their service lives and/or have served beyond their originally intended design lives. Proper evaluation of existing bridges for their continuing services is a very important and challenging task in bridge engineering across the world.

There are several fundamental differences between evaluation of existing bridges and design of new bridges. First of all, bridge evaluation deals with an existing structure of a specific physical condition, at a specific time, and for specific loads, whereas bridge design produces construction documents, including material specifications and structural fabrication/erection details, for constructing a new structure for codified combinations of design loads. Second, load distribution in an existing bridge is determined by the as-built condition of the structural system, as well as the actual configuration and positions of travel lanes and/or rail tracks; all structural components, primary and secondary, participate in load sharing to varying degrees. In new bridge design, however, load distribution among structural components is based on provisions of design specifications and the design loads are assumed to be carried by primary members only, in certain prescribed manners. Third, actual connections and bearings of existing bridges rarely behave as idealized pins, hinges, rollers, or fixed supports as is assumed in the original design analysis. Last, the effect of time on material strength is not quantitatively accounted for in bridge design, but it plays a significant role in bridge evaluation regarding the remaining strength of materials and structural components after aging and deterioration, the performance of connections and supports over time, as well as the variation of loading frequency and magnitudes over time.

Accurate assessment of load effects and strengths of structural components is more important in bridge evaluation than in bridge design because its impact is more consequential in terms of decisions in bridge maintenance, weight postings, repairs, strengthening, and ultimately replacement. In bridge evaluation, site-specific information on loads and physical condition of the structure at the time of evaluation is critical to accurately assess the demands versus the load-carrying capacity of the bridge.

To accurately evaluate existing steel bridges, a good understanding of the following information is necessary:

- Expected loads and their combinations at the bridge site
- Physical conditions of the structure at the time of evaluation (corrosion, section losses, etc.)
- Performance of connections and bearings
- Fatigue-prone details
- Susceptibility to fracture
- Redundancy of the structural system
- Load–force/stress correlations in critical members and connections

14.2 General Scope, Methods, and Specifications

14.2.1 General Scope of Steel Bridge Evaluation

Steel bridges have many different structural types, including rolled I-beams, welded plate girders, trusses, arches, cable-stayed, and suspension. One thing all bridge structures have in common is to support a deck, either concrete or steel orthotropic, for vehicles and/or pedestrians or rails for trains. The primary function of all bridge structures is to safely transfer the expected loads to their foundations without distress or excessive deformations or vibrations throughout the structures. Therefore, bridge evaluation should address all structural components, members, and connections along entire load paths.

For most types of bridges, the general load path for live loads can be extracted as deck or rails → floor system (stringers/floor beams); connections → cables/hangers and connections, as applicable → main members (beams, girders, trusses, arches, main cables, etc.); and connections → bearings and/or anchorages → substructure and foundation.

The primary objective of bridge evaluation is to quantitatively assess all structural members and connections along entire load paths for all possible modes of failure in accordance with a governing code. For existing steel bridges, check items typically include the following for all structural components in the physical condition at the time of evaluation:

- Tension
- Shear
- Compression
- Bearing
- Fatigue
- Fracture

14.2.2 Methods for Bridge Evaluation

Several methods are available for evaluating the condition and/or structural integrity of existing bridges for required loads, varying from the following:

- Simplified methods as prescribed in bridge design/evaluation specifications
- Refined analysis methods (e.g., finite element modeling) for load effects
- Field load testing/measurements for actual load–response correlations
- Material sampling/testing for chemical and mechanical properties
- Nondestructive testing/evaluation (NDT/E) for detection of deficiencies

The selection of methods to be employed should be based on the requirements of specific bridge evaluation needs. In any case, bridge evaluation must be in compliance with governing specifications, depending on the locality and function (highway, railway, or pedestrian) of the bridge.

The acceptance criteria in bridge evaluation are generally consistent with those of bridge design based on one of the following philosophies:

- Allowable Stress method
- Load Factor method
- Load and Resistance Factor method

14.2.3 Specifications for Steel Bridge Evaluation and Rating

Existing highway bridges are evaluated in accordance with the American Association of State Highway and Transportation Officials (AASHTO) *Manual for Bridge Evaluation* (MBE) (AASHTO 2013), which was first published in 2008 based on a combination of the AASHTO *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* (AASHTO 2003) and the AASHTO *Manual for Condition Evaluation of Bridges* (AASHTO 1994). The AASHTO MBE contains the following sections: 1: Introduction; 2: Bridge Files (Records); 3: Bridge Management Systems; 4: Inspection; 5: Material Testing; 6: Load Rating; 7: Fatigue Evaluation of Steel Bridges; 8: Nondestructive Load Testing; and Appendix A: Illustrative Examples. Sections 5 through 8 provide provisions and methods for the evaluation and rating of existing steel bridges.

For railway bridges, the American Railway Engineering and Maintenance-of-way Association (AREMA) *Manual for Railway Engineering* (MRE) (AREMA 2012), provides recommended practice for design, fabrication, and construction of new bridges as well as inspection, rating, and maintenance of existing bridges. AREMA MRE Chapter 15, "Steel Structures," Part 7, "Existing Bridges," contains the following sections: 7.1: General; 7.2: Inspection; 7.3: Rating; 7.4: Repair; Strengthening and Retrofitting; and 7.5: Maintenance.

14.2.4 Connections versus Members

All primary members and connections must be quantitatively evaluated in accordance with applicable criteria as required at the time of evaluation. It should be noted that different design criteria exist for members and connections depending on the age of a bridge and the design specifications used.

Highway bridge connections were historically designed to the full strength of the members connected. In the 1931 first edition of American Association of State Highway Officials (AASHO) *Standard Specifications for Highway Bridges and Incidental Structures* (AASHO 1931), Article 5.6.29—Strength of Connections—states: "Unless otherwise provided, connections shall be proportioned to develop the full strength of the members connected." Article 5.6.30—Splices—says: "… Splices, whether in tension or compression, shall be proportioned to develop the full strength of the members spliced and no allowance shall be made for the bearing of milled ends of compression members. …" Article 5.6.33—Gusset

Plates—states: "… The gusset plates shall be of ample thickness to resist shear, direct stress, and flexure, acting on the weakest or critical section of maximum stress. …"

However, the full-strength connection design requirements were revised in the 1941 third edition of AASHO *Standard Specifications for Highway Bridges* (AASHO 1941): Article 5.6.30—Strength of Connections—states: "Except as otherwise provided herein, connections shall be designed for the average of the calculated stress and the strength of the member, but they shall be designed for not less than 75 percent of the strength of the member." Article 5.6.31—Splices—says: "… Splices, whether in tension, compression, bending, or shear, shall be designed for the average of the calculated stress and the strength of the member, or section, but they shall be designed for not less than 75 per cent of the strength of the member. …" Article 5.6.34—Gusset Plates—states: "… The gusset plates shall be of ample thickness to resist shear, direct stress, and flexure, acting on the weakest or critical section of maximum stress. … If the unsupported edge of a gusset plate exceeds the following number of times its thickness, the edge shall be stiffened …" Such connection design criteria have remained essentially the same through the latest 17th edition of AASHTO *Standard Specifications for Highway Bridges* (AASHTO 2002) and the 6th edition of AASHTO *LRFD Bridge Design Specifications* (AASHTO 2012).

For the design of railway steel bridges, AREMA MRE (AREMA 2012) Chapter 15, "Steel Structures," Section 1.5.9, Connections and Splices, states: "Splices of main members shall have a strength not less than the capacity of the member and … End connections of main members receiving load from the combined effect of floor system and truss action shall have a strength not less than the capacity of the member. End connections of members carrying direct load from one floor beam only shall be proportioned for at least 1.25 times their computed reactions."

For gusset plates at the main connections of steel truss bridges, the Federal Highway Administration (FHWA) of the United States Department of Transportation (USDOT) issued special guidelines in 2009 (USDOT FHWA 2009) in light of the collapse of the I-35W Mississippi River Bridge in Minneapolis, Minnesota, on August 1, 2007. The guidelines addressed concerns on load-carrying capacities of gusset plates in non-load-path-redundant steel truss bridges.

14.3 Safety Margin and Reliability Index

14.3.1 Measurement of Safety Margin and Reliability Index

A bridge is considered to be safe if its structural strength, or resistance, is greater than the structural demand, or load effect, from the loads it is expected to carry. In the simplest way, a structure is safe if the following equation is satisfied in all primary load-carrying members and connections for all possible failure modes (flexure, shear, axial loading, etc.):

$$R(t) > Q(t) \tag{14.1}$$

where t is time from the beginning of service; $Q(t)$ is the load effect, such as bending moment or shear force, in a member or connection due to combined loads under consideration at time t; and $R(t)$ is resistance of the member or connection corresponding to $Q(t)$ at the same time t.

A structure may become unsafe when the load effect, $Q(t)$, in a primary member or connection exceeds the corresponding resistance, $R(t)$, at any time. Figure 14.1 graphically illustrates the basic principles of probabilistic design as well as the probability of failure, p_f.

In Figure 14.1, both load effect Q and member resistance R are assumed to follow the normal distribution, where $\overline{Q}$ is the mean value of load effect Q, σ_Q is the standard deviation of load effect Q, $\overline{R}$ is the mean value of structural resistance R, σ_R is the standard deviation of structural resistance R, and p_f is the probability of failure (when $Q \geq R$).

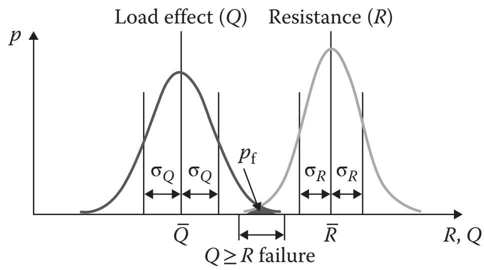

FIGURE 14.1 Illustration of structural safety measurement.

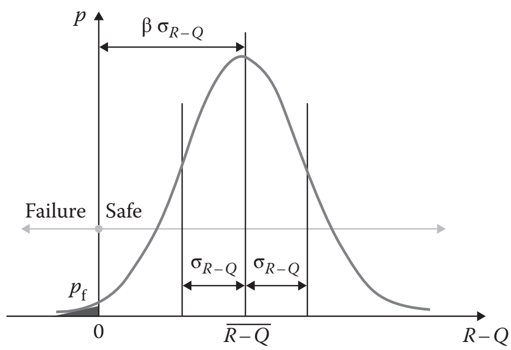

FIGURE 14.2 Illustration of structural reliability index.

Bridge safety margin is generally measured as the probability for the member resistance to exceed the load effect, that is,

$$\text{Safety margin} = 1 - p_f \tag{14.2}$$

The aforementioned bridge safety margin can be quantified by one random variable that measures the difference between structural resistance and load effect, $R - Q$. Thus, the probability of failure corresponds to the occurrence of $R - Q$ less than zero (see Figure 14.2):

$$p_f = \text{Probability}\ \{R - Q < 0\} \tag{14.3}$$

The shaded area for p_f in Figure 14.2 is equivalent to the shaded area for p_f in Figure 14.1.

The primary objective of bridge design is to size the structural members and connections properly so that structural resistance R is sufficiently, but not overly, higher than load effect Q. In probabilistic design, the safety margin is based on a predetermined, acceptable level of probability of failure p_f and the design parameters (design loads, member design strengths, etc.) are determined based on statistical properties of R and Q, such as $\bar{Q}$, σ_Q, $\bar{R}$, and σ_R.

In bridge design, reliability index, β, is a term used to measure the safety margin in lieu of the probability of failure, p_f. If the variable $R - Q$ can be represented by the normal distribution, reliability index becomes the ratio of the mean to the standard deviation of $R - Q$, that is,

$$\beta = \frac{\overline{R - Q}}{\sigma_{R-Q}} \tag{14.4}$$

As shown in Figure 14.2, the objective of probabilistic design is to place the distribution curve of $R - Q$ properly so that probability of failure p_f meets the predetermined level of acceptance. For normal

TABLE 14.1 Corresponding Values of β and p_f

β	p_f
1.0	1.59×10^{-1}
2.0	2.3×10^{-2}
2.5	6.2×10^{-3}
3.0	1.3×10^{-3}
3.5	2.33×10^{-4}
4.0	3.17×10^{-5}

distributions, reliability index β represents the number of standard deviations, σ_{R-Q}, for the distance between the origin and the mean, $\overline{R-Q}$, at the center of the distribution curve. The higher the β, the lower the p_f.

Table 14.1 lists the corresponding values of reliability index β and probability of failure p_f for normal distributions. In AASHTO *LRFD Bridge Design Specifications* (AASHTO 2012), a reliability factor of 3.5 is used, which corresponds to a probability of failure of .000233.

For different normal distribution curves to satisfy a same β or p_f requirement, the mean, $\overline{R-Q}$, needs to vary depending on the magnitude of the standard deviation σ_{R-Q}. A wider distribution with a greater σ_{R-Q}, or greater dispersion, requires a greater mean value of $R - Q$ to satisfy the same β or p_f requirement. This indicates that for the same mean design load, greater uncertainties in R and Q would require higher mean structural resistance and, thus, result in designs with larger member and connection sizes.

14.3.2 Uncertainties in Bridge Evaluation

In bridge evaluation, structural failures may be defined as combinations of loads causing high forces or stresses that exceed member or material strength in at least one primary member or connection. Therefore, safety evaluation of existing bridges usually includes the following general tasks:

- Select suitable bridge evaluation specifications and safety criteria.
- Determine the magnitudes, variations, and combinations of loads, and compare them with the original design load.
- Identify critical structural components (members and connections) and failure modes that may govern the safety of the bridge.
- Establish the load–force and/or load–stress correlations in the critical structural components.
- Assess the physical conditions of the critical structural components at the time of evaluation, and compare them with the original conditions documented in as-built plans.
- Evaluate the critical components for governing failure modes under required load combinations based on current structural conditions per governing criteria.

Bridge evaluation deals with existing bridges that were designed using codes that may or may not be probability and statistics based. It is important to acquire and review the original design documents to understand the design method, loads, material properties, and key construction issues, although such information is not always available.

There are many uncertainties and variations that affect both the load effects and the structural strength of existing bridges. The following factors contribute to the variation of load effects, $Q(t)$, at a given time:

- Variation of truck weights and traveling patterns
- Variation of other simultaneous loads such as dead loads, temperatures, and winds
- Uncertainties in load–stress correlations

- Uncertainties in the condition of supports/bearings and expansion joints
- Changes of all of the aforementioned factors over time

The following factors contribute to variations of structural resistance, or member capacities, $R(t)$, at a given time:

- Variation of member sizes and geometry
- Variation of material properties
- Uncertainties in construction quality
- Uncertainties in redundancy of the structural system
- Structural and material degradations over time

14.3.3 Methods for Improving Accuracy of Bridge Evaluation

As illustrated in Figures 14.1 and 14.2, bridge safety margin, and the probability of failure, is determined by the statistical distributions of structural resistance R and load effect Q. For the evaluation of existing bridges, therefore, better assessment of the statistical characteristics of R and Q at a given time ($\bar{Q}$, σ_Q, $\bar{R}$, and σ_R) helps to yield more accurate safety measurements.

Based on the previous discussions in Sections 14.3.1 and 14.3.2, the following three measures help to improve the accuracy of bridge evaluation:

- Refine load models by measuring site-specific truckload histograms through techniques such as weigh in motion (WIM).
- Refine load–stress correlations in critical structural components through finite element analysis and/or field load testing.
- Determine in situ conditions, visible and nonvisible, of critical components and materials by performing NDT/E.

The objective of these measures is to reduce the width, or dispersion, of distribution curves, as illustrated in Figure 14.3. Site-specific load models and refined load–stress correlations should reduce uncertainties and thus reduce the dispersion of Q compared with calculating the load effect from the design load model using the simplified method. Similarly, NDT/E results provide site-specific information for a more accurate assessment of structural resistance, thus reducing the dispersion of R compared with what is provided by the code formulas.

Figure 14.3 illustrates the advantages of bridge evaluation using refined methods. Typically, structural resistance R tends to decrease and nominal load effect Q tends to increase over time during a bridge's service life. Therefore, the difference between their mean values, $\bar{R}$ and $\bar{Q}$, tends to decrease over time. However, refined analysis methods can help reduce the dispersions of both R and Q and thus better estimate the probability of failure, p_f. Since design specifications are for general applications and are usually based on relatively higher dispersions for both R and Q to include some worst conditions, most

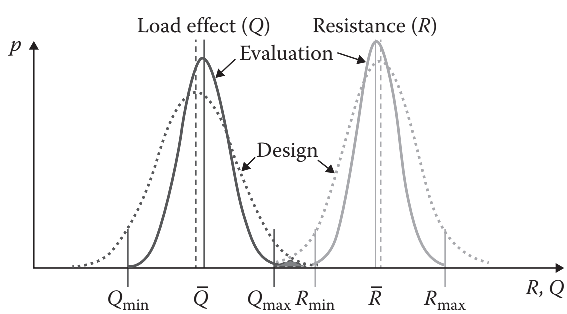

FIGURE 14.3 Illustration of benefits from refined analysis methods.

structural designs can be characterized as "conservative." As a result, site-specific information resulting from refined measures in bridge evaluation can usually yield a lower probability of failure, or higher reliability index, compared with what is given by the design specifications.

It should be noted that the determination of site-specific statistical distributions of R and Q is practically prohibitive for most bridge evaluation projects, However, the site-specific upper limit of load effects, Q_{max}, and lower limit of structural resistance, R_{min}, can be estimated using the aforementioned advanced methods, as shown in Figure 14.3. Once the probabilities of exceeding these limits are estimated, the bridge safety margin, or the reliability index, can be estimated more accurately than by using the general formulas in the code specifications.

The employment of advanced analysis methods for bridge evaluation is allowed and their benefits are recognized with proper reliability factors in AASHTO MBE (AASHTO 2013). Various methods of advanced analysis and nondestructive field testing/measurements can be used in load rating, fatigue evaluation, or assessment of nonvisible structural conditions using NDT/E technologies.

14.4 Load Rating

14.4.1 General Concept and Definition

Load rating measures a bridge's load-carrying capacity for specific loading vehicles to maintain its safe use. Typically, load rating of a bridge consists of a set of rating factors with a baseline value of 1.0 or ratings in gross vehicle weight, one for a specific type of rating vehicle. Load ratings of a bridge provide a basis for making decisions on weight limit posting, overweight vehicle permits, and structural strengthening or replacement of the bridge. Load ratings should reflect the physical condition of the bridge at the time of evaluation and should be updated whenever changes are identified in the structural condition or the dead load. Bridge load ratings are reported to the National Bridge Inventory maintained by USDOT FHWA and other bridge management systems.

All primary members and connections of the bridge should be load rated for possible modes of failure, and the lowest rating value becomes the load rating of the bridge for each vehicle type. The general concept of bridge load rating is defined as follows:

$$\mathrm{RF} = \frac{C - D}{\mathrm{L} + I} \tag{14.5}$$

where RF is the rating factor of a member or connection for a specific rating vehicle, C is the capacity of the member or connection for a specific failure mode, D is the effect of total dead loads, and $(\mathrm{L} + I)$ is the effect of the rating vehicle plus impact.

Therefore, $\mathrm{RF} \geq 1.0$ represents sufficient capacity and $\mathrm{RF} < 1.0$ represents insufficient capacity for the bridge to carry the rating vehicle. The magnitudes of C, D, and $(\mathrm{L} + I)$ in the rating calculation include load and resistance factors as well as other reliability factors as applicable. They vary depending on the rating criteria (Allowable Stress, Load Factor, or Load and Resistance Factor method) as well as the method of analysis (simplified analysis, finite element analysis, or field load testing).

Once the rating factor, RF, is determined, load rating in vehicle weight is calculated as: load rating = (RF)(vehicle gross weight) for each type of rating vehicle.

Generally, only permanent loads and vehicular loads are considered in load rating. Permanent loads include dead loads of existing condition at the time of analysis (future wearing surface need not be included) and locked-in force effects from construction (e.g., posttensioning forces). Environmental loads such as wind, ice, temperature, stream flow, and earthquake are usually not considered in load rating except when unusual conditions warrant their inclusion. However, the effects of wind on special structures such as movable bridges, long-span bridges, and other high-level bridges should be considered per applicable standards.

The following key issues should be taken into consideration in bridge load rating:

- Changes in existing structural conditions, material properties, loads, and traffic conditions at the bridge site
- Structural deteriorations including, but not limited to, cracking, section loss, degradation of steel–concrete bond/anchorage, changes in structural geometry, and missing fasteners
- Results of the most recent inspection and intervals of future inspections
- Engineering judgments in determining structural conditions and arriving at posting/permit/repair/replacement decisions.

14.4.2 Methods for Bridge Load Rating

There are two general approaches for determining bridge load rating:

1. Analytical calculations using the simplified or refined method.
2. Experimental evaluation using nondestructive field testing of various types, by choice (for improving accuracy) or not (when lack of necessary information for analytical rating).

Similar to design, bridge load rating can be performed using one of the three evaluation criteria (safety and serviceability standards), depending on the practice of the bridge owner:

1. Allowable Stress method
2. Load Factor method
3. Load and Resistance Factor method

Bridge load rating using any of the methods should be based on as-built member properties and results of a recent field inspection for consideration of deterioration and section loss.

14.4.3 Load Rating of Highway Bridges

AASHTO MBE (AASHTO 2013) sets forth criteria and procedures for the load rating of existing highway bridges in Section 6, Load Rating. The MBE contains provisions for three different load rating methods: Allowable Stress Rating (ASR), Load Factor Rating (LFR), and Load and Resistance Factor Rating (LRFR). No preference is placed on any rating method and any one of them may be used to establish live load capacities for load posting.

14.4.3.1 AASHTO LRFR Load Rating Method

The LRFR procedure provides a rating method that is consistent with the Load and Resistance Factor Design (LRFD) design philosophy, which is reliability and statistics based and more scientific than the ASR and LFR procedures. The general LRFR load rating equation is Equation 14.6, which is to be checked at each applicable limit state for each member/connection subject to a single force effect for each possible failure mode (flexure, shear, or axial). The lowest rating factor, RF, value controls a bridge's load rating:

$$\mathrm{RF} = \frac{C - \gamma_{\mathrm{DC}}(\mathrm{DC}) - (\gamma_{\mathrm{DW}})(\mathrm{DW}) \pm (\gamma_P)(P)}{(\gamma_{\mathrm{LL}})(\mathrm{LL}+\mathrm{IM})} \tag{14.6}$$

where C is capacity $= \varphi_c\varphi_s\varphi R_n$ for the strength limit states ($\varphi_c\varphi_s \geq 0.85$) and C is f_R for the service limit states; φ_c is condition factor, φ_s is system factor, and φ is LRFD resistance factor; R_n is nominal member resistance (as inspected) and f_R is allowable stress specified in the LRFD code; DC is dead load effect due to structural components and attachments, DW is dead load effect due to wearing surface and utilities, P refers to permanent loads other than dead loads, LL is live load effect, and IM is dynamic load allowance; γ_{DC} is LRFD load factor for structural components and attachments, γ_{DW} is LRFD load factor for wearing surface and utilities, γ_P is LRFD load factor for permanent loads other than dead loads, and γ_{LL} is evaluation live load factor.

It should be noted that LRFR does not contain load modifiers (for ductility, redundancy, and operational importance) as in LRFD but instead uses the condition and system factors, φ_c and φ_s. For bridge load rating, strength is the primary limit state and service and fatigue limit states are selectively applied.

The LRFR load factors for commonly used structural types and limit states are summarized in Table 14.2.

The LRFR condition factor, φ_c, as defined in Table 14.3, accounts for increased uncertainty in the resistance of deteriorated members at the time of rating and between future inspections.

The system factor, φ_s, as defined in Table 14.4 for flexural and axial effects ($\varphi_s = 1.0$ for shear), accounts for redundancy of the structural system—the capability to carry loads after damage or failure of one or more of its members.

LRFR load rating procedures include three different evaluation criteria, or reliability levels, of varying live load models:

1. Design load rating (first-level evaluation)
2. Legal load rating (second-level evaluation)
3. Permit load rating (third-level evaluation)

LRFR Design load rating is based on LRFD HL-93 design loading and is the first-level assessment in the screening process to identify the need for further evaluation for Legal and Permit load rating.

TABLE 14.2 AASHTO LRFR Limit States and Load Factors for Load Rating

Bridge Type	Limit State[a]	Dead Load γ_{DC}	Dead Load γ_{DW}	Design Load: Inventory γ_{LL}	Design Load: Operating γ_{LL}	Legal Load γ_{LL}	Permit Load γ_{LL}
Steel	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-l and 6A.4.4.2.3b-l	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-l
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
	Fatigue	0.00	0.00	0.75	—	—	—
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-l and 6A.4.4.2.3b-l	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-l
	Service I	1.00	1.00	—	—	—	1.00
Prestressed Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-l and 6A.4.4.2.3b-l	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-l
	Service III	1.00	1.00	0.80	—	1.00	—
	Service I	1.00	1.00	—	—	—	1.00
Wood	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-l and 6A.4.4.2.3b-l	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-l

Notes:
- Shaded cells of the table indicate optional checks.
- Service I is used to check the 0.9 F stress limit in reinforcing steel.
- Load factor for DW at the strength limit state may be taken as 1.25 where thickness has been field measured.
- Fatigue limit state is checked using the LRFD fatigue truck (see Article 6A.6.4.1).

[a] Defined in AASHTO *LRFD Bridge Design Specifications*.

TABLE 14.3 AASHTO LRFR Condition Factor, φ_c

Structural Condition of Member	φ_c
Good or satisfactory	1.00
Fair	0.95
Poor	0.85

TABLE 14.4 AASHTO LRFR System Factor, φ_s, for Flexural and Axial Effects

Superstructure Type	φ_s
Welded members in two-girder/truss/arch bridges	0.85
Riveted members in two-girder/truss/arch bridges	0.90
Multiple eyebar members in truss bridges	0.90
Three-girder bridges with girder spacing of 6 ft.	0.85
Four-girder bridges with girder spacing ≤ 4 ft.	0.95
All other girder bridges and slab bridges	1.00
Floor beams with spacing >12 ft. and noncontinuous stringers	0.85
Redundant stringer subsystems between floor beams	1.00

The Design load rating is calculated at two rating levels:

1. Inventory, with the same level of target reliability index $\beta = 3.5$ based on a severe load case of 5000 average daily tuck traffic (ADTT)
2. Operating, of lowered target reliability index of approximately $\beta = 2.5$ to reflect reduced exposure period, site realities, and economic considerations of rating versus design

Bridges that pass the Inventory Design load check (RF ≥ 1) will have adequate capacity for all AASHTO and state legal loads within the LRFD exclusion limits. No further evaluation is necessary. Bridges passing only the Operating Design load check will have adequate capacity for all AASHTO legal loads but may not rate (RF < 1) for all state legal loads. For bridges not passing the Operating Design load rating (RF < 1), Legal load rating is required.

LRFR Legal load rating is the second-level load rating in the screening process and provides a single safe load capacity for a given truck configuration applicable to AASHTO and state legal loads. The live load factors are specified in MBE based on vehicle type and truck traffic volume ADTT at the bridge site. Dynamic load allowance may be used as the LRFD design value, reduced based on the approach and deck surface conditions, or determined from field testing. The load and resistance factors are calibrated per the Operating level of reliability for redundant bridges in good condition, and adjustments are allowed per site-specific data via φ_c and φ_s. Legal load rating provides a basis for decision making related to load posting or bridge strengthening. Only bridges that pass the Legal load rating ($RF > 1$) should be evaluated for overweight permits.

LRFR Permit load rating is the third-level load rating and only for bridges with sufficient capacity for legal loads (RF > 1.0). This procedure is for the evaluation of permits to allow the passage of vehicles above the legally established weight limitations on the highway system, sometimes along specified routes. Permits may be for a single trip, for multiple trips, or on an annual basis for unlimited trips, for a specified gross weight and axle/weight configuration. Permit vehicles may be allowed to mix with normal traffic or to be escorted in a manner to control their speed, lane position, the presence of other vehicles, or some combination thereof. In Permit load rating, live load is the actual permit vehicle weight and axle configuration, with load factors defined in LRFR based on permit type, loading condition, and site traffic data. Similar to Legal load rating, dynamic load allowance may be used as the LRFD design value, reduced based on the approach and deck surface conditions, or determined from field testing. A small likelihood of multipresence is assumed, and serviceability criteria are also to be checked.

14.4.3.2 AASHTO ASR and LFR Load Rating Methods

In addition to the LRFR load rating method described in Part A, MBE (AASHTO 2011) also provides ASR and LFR methods in Part B as choices, with no preference placed on any method. MBE states that bridge owners should implement standardized procedures for bridge load rating. Although LRFR provides uniform reliability in bridge load ratings and load postings, ASR and LFR may be especially useful for comparison with past practices. Matters not covered in MBE regarding ASR and LFR are governed by AASHTO standard specifications (AASHTO 2002).

ASR and LFR each has two rating levels: the Inventory rating at the design level of safety for infinite use and the Operating rating at a reduced level of safety for limited use. The general load rating equation for ASR and LFR is as follows:

$$\mathrm{RF} = \frac{C - A_1 D}{A_2 L(1+I)} \tag{14.7}$$

where D is the effect of total dead load, L is the live load effect of the rating vehicle, and I is the dynamic impact of the rating vehicle. For ASR, C is allowable stress varying with rating level (specified in MBE, Tables 6B.5.2.1–1 and 6B.5.2.1–2) and $A_1 = A_2 = 1.0$. For LFR, C is nominal capacity for both Inventory and Operating levels; $A_1 = 1.3$ for both Inventory and Operating levels; and $A_2 = 2.17$ and 1.3 for Inventory and Operating levels, respectively.

Posting is generally based on the Operating level of ASR and LFR but varies with agencies. The ratio between the Operating rating and the Inventory rating is generally $\mathrm{RF_{OPR}/RF_{INV}} \approx 1.36$ for ASR and $\mathrm{RF_{OPR}/RF_{INV}} = 1.67$ for LFR.

14.4.3.3 Load and Resistance Factor Rating Provisions for Load Rating of Steel Bridges

Where possible, steel properties used in load rating should be based on the construction documents per the ASTM or AASHTO designation and grade for minimum yield and tensile strengths of the structural steel. For structural steels without specifications, minimum strengths may be assumed as per Table 14.5.

If necessary, coupon tests can be performed for steel properties. In this case, nominal values for yield and tensile strengths are typically taken as the mean test value minus 1.65 times the standard deviation for a 95% confidence limit. Guidance on material sampling for bridge evaluation is provided in Article 5.3 of the MBE. Actual values of mill reports should not be used, but rather the minimum values should be used in bridge load rating.

LRFR load rating of steel bridges shall follow the applicable limit states and load combinations specified in Table 6A.4.2.2–1 of the MBE, varying with load rating procedures. For strength limit states, resistance factor φ values are as specified in LRFD Design Article 6.5.4.2. For the service limit state, $\varphi = 1.0$. The following limit states shall be checked for load rating of steel bridges:

Limit states for design load rating:

- Strength I
- Service II (for control of permanent deflection)
- Fatigue I for details of category C or lower (fatigue II optional)

TABLE 14.5 AASHTO MBE–Suggested Minimum Mechanical Properties of Structural Steel by Year of Construction

Year of Construction	Minimum Yield Point or Minimum Yield Strength, F_y (ksi)	Minimum Tensile Strength, F_u (ksi)
Prior to 1905	26	52
1905–1936	30	60
1936–1963	33	66
After 1963	36	66

Limit states for Legal load rating:

- Strength I
- Service II

Limit states for Permit load rating:

- Strength II
- Service II

One important issue in load rating steel bridges is to evaluate the impact of steel deterioration and corrosion. A deteriorated structure may behave differently from what is intended by design, and different failure modes may govern its load capacity. Corrosion is the major cause of deterioration in steel bridges. Effects of corrosion include section loss, unintended fixities, movements and pressures, and reduced fatigue resistance.

For load rating purposes, the residual capacity of structural members may be computed as the product of a percent effective, p_{eff}, and the code-based capacity calculated from the intact condition to account for the actual condition of the member due to deterioration. The term p_{eff} may be estimated at the time of field inspection and should be specific for different possible failure modes. For example, $p_{eff} = 0.95$ for flexural capacity of steel beams that exhibit corrosion and insignificant section loss.

14.4.4 Load Rating of Railway Bridges

The criteria for load rating of steel railway bridges are provided in Part 7, "Existing Bridges," in Chapter 15, "Steel Structures," of AREMA MRE (AREMA 2012). Section 7.3, Rating, specifies two rating levels: normal rating and maximum rating. Rating of railway bridges is performed based on allowable stresses.

Normal rating is the load level that can be carried by the existing structure for its expected service life. The rating is dependent on a specified speed, as impact reductions are allowed for reduced speeds. Load rating of steel railway bridges are based on the allowable stress method, and the allowable stresses for normal rating are the same as those specified for design of new bridges in Table 15–1–11, Basic Allowable Stresses for Structural Steel, Rivets, Bolts and Pins, in AREMA MRE (AREMA 2012). For example, for the stress calculated based on gross section of axial tension members made of structural steels, the allowable stress for normal rating is specified as $0.55F_y$.

Maximum rating is the load level that the structure can support at infrequent intervals, with any applicable speed restrictions. Allowable stresses for maximum rating are specified in Table 15–7–1, Allowable Stresses for Maximum Rating, in the AREMA MRE (AREMA 2012). For example, for the stress calculated based on gross section of axial tension members made of open-hearth steels and high-performance steels, the allowable stress for maximum rating is specified as $0.8F_y$, which is 1.455 times the corresponding allowable stress for normal rating.

14.4.5 Load Rating through Nondestructive Field Load Testing and Three-Dimensional Computer Modeling

Engineering experience has indicated that conventional rating calculations based on the simplified structural analysis method may yield inaccurate, and in many cases overly conservative, bridge load ratings. This problem becomes more pronounced for aged or deteriorated structures or for structures lacking information on material properties or construction plans. Field testing helps to understand the actual structural behavior under load and identify and quantify the inherent load-carrying mechanisms that are not considered in conventional analysis. These mechanisms include unintended composite action between the main load-carrying members and the secondary components such as deck/floor components, barriers, and lateral/sway bracing systems; actual live load distribution among multiple beams or trusses; and the influence of support conditions that result in unintended continuity/fixity.

On the other hand, for bridges that have hidden structural deficiencies or deteriorations that are not apparent to visual inspections field-measured deflections and strains may reveal such problems.

To better understand the load-carrying capacities of bridge structures for public safety and for more efficient asset management, bridge load rating through field testing has proved to be a more accurate and cost-effective technique. Bridge evaluation through load testing often results in improved load ratings since, typically, the live load effects are found to be lower than those predicted by conventional analysis methods. Engineering experience has demonstrated that in many cases weight limits based on conventional rating calculations may be raised or eliminated altogether.

Procedures for bridge load rating through field load testing were developed in National Cooperative Highway Research Project (NCHRP) Project 12–28(13)A and reported in NCHRP Research Results Digest No. 234, *Manual for Bridge Rating Through Load Testing* (NCHRP 1998). AASHTO officially adopted the approach of bridge rating through load testing in the 2003 *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* (AASHTO 2003), which became part of the first edition of the MBE in 2008 (AASHTO 2013). Two general procedures have been prescribed in Section 8 of the MBE for bridge load testing: diagnostic load testing and proof load testing. Either method, as appropriate for individual circumstances, is able to produce more accurate bridge load ratings.

Diagnostic load testing determines the actual responses of key structural components, in terms of measured strains and deflections, to test vehicles of known weights and axle configurations crossing the bridge at controlled locations and speeds. The MBE provides guidelines and formulas for determining the rating factor from adjusting a theoretical rating factor based on field strain measurements from load testing. In practice, a three-dimensional (3-D) finite element model is often established based on best available information and compared with the load test results. After being adjusted or calibrated with field measurements, the model is used to determine the maximum load effects for dead load and all required rating vehicles for rating calculations. In bridge load rating through diagnostic load testing, the test load is usually at the service load level and member capacities are calculated based on section and material properties specified in the bridge plans. The rating calculations typically yield inventory and operating ratings or legal or permit ratings based on the load rating method to be employed.

Alternatively, proof load testing physically verifies the lower bound load-carrying capacity of a bridge by placing a predetermined target test load in an incremental manner while the structure is closely monitored and responses are measured by strain and displacement sensors during the multiple-step loading and unloading process. The target test load is generally 1.4 times the service load, although it may range between 1.3 and 2.2 times the rating vehicle weight (details provided in MBE). Proof load testing usually requires no load rating calculations and concludes whether the operating rating factor exceeds 1.0 for a specific rating vehicle.

Generally, a bridge is first rated using simplified structural analysis methods with the load distribution factors as prescribed in the specifications, which tend to be conservative. If load ratings from the simplified methods are insufficient for bridge service requirements, refined methods such as the finite element analysis may be used for improved accuracy. If the refined analytical models are still believed to not accurately represent the true behavior and load distribution of the structure and its components, field load testing is justified for more accurate bridge load ratings.

A general procedure for evaluation or reevaluation of bridge load ratings can be summarized as follows:

1. Review available information including, but not limited to, plans, inspection reports, and previous rating calculations to understand the condition of the structure as well as the critical elements and failure modes that govern previous load ratings. The review should also verify key dimensions, existence and degree of deterioration or section loss, and compliance of previous rating calculations with the current code.
2. Assess prospective benefits of a refined analysis, such as unintended composite action of the deck-beam or floor-truss system, participation of secondary components (parapets, sidewalks, bracing members, etc.), and broader live load distributions than those assumed by design/rating specifications.

3. Perform a baseline rating analysis for all rating vehicles using a 3-D finite element model including all primary and secondary structural components. The modeling of connections, joints, and bearings should consider both their originally intended performance and their actual condition at the time of load rating. The baseline model is used to determine the maximum force effects of total dead load and all rating vehicles and to determine critical sections that govern the load rating.
4. Review the results of the refined load rating analysis from the baseline finite element model with the assumptions, and determine whether it is necessary to proceed with field load testing; if yes, select the type of field testing (diagnostic or proof load) depending on the bridge condition and the load rating need.
5. Develop an instrumentation plan for the placement of strain gages and displacement transducers based on the results of the baseline rating analysis. Also develop a fieldwork plan including access equipment or scaffolding as necessary; maintenance of traffic; the number, type, and weight of test trucks; and the procedure of test runs.
6. Install sensors on the structure per the instrumentation plan and perform load testing using test trucks while responses from all the sensors are recorded by a data acquisition system. Critical test runs should be repeated for verification purposes, and recorded data should be closely monitored for their maximum magnitudes and zero return after unloading. Test runs should include single truck crossings at crawl and full speed at various lateral positions and two, or more, trucks crossing side by side.
7. Adjust and refine the baseline finite element model using the test results. Certain features of the model, such as concrete stiffness, properties of connections between structural components, properties and connections of secondary members, and bearing or support conditions, can be modified to achieve better agreement with measured structural responses of the test runs.
8. Calculate refined rating factors using the adjusted computer model for all the rating vehicles, and prepare a summary report describing the procedure and results of the refined analysis and load testing as well as recommended load ratings for the rating vehicles.

14.5 Fatigue Life Assessment and Crack Evaluation

14.5.1 Background of AASHTO and AREMA Fatigue Specifications

14.5.1.1 AASHTO Fatigue Specifications for Highway Bridges

Historically, several AASHTO specification documents have presented fatigue provisions for the design and evaluation of steel highway bridges. Listed in the chronological order of their initial publication, these specification documents are as follows:

- *Standard Specifications for Highway Bridges* (AASHTO 2002)—"The Standard Specifications"
- *Guide Specifications for Fatigue Evaluation of Existing Steel Bridges* (AASHTO 1990)—"The Fatigue Guide Specs"
- *LRFD Bridge Design Specifications* (AASHTO 2012)—"The LRFD"
- *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* (AASHTO 2003)—"The LRFR"
- *The Manual for Bridge Evaluation* (AASHTO 2013)—"The MBE"

Intended primarily for the design of new bridges, "The Standard Specifications" had been the dominating fatigue code in the United States for the design and evaluation of steel highway bridges until the 1990s when other specification documents started becoming available. "The Fatigue Guide Specs," published in 1990, were developed based on NCHRP Report No. 299 (Moses et al. 1987) from NCHRP Project 12–28(3), "Fatigue Evaluation Procedures for Steel Bridges," initiated in 1985. "The Fatigue Guide Specs" and "The LRFR" were developed exclusively for the purpose of evaluating existing steel bridges, and both allow several alternative methods that are not available in "The Standard Specifications" or "The LRFD." These methods include field strain measurements at the fatigue details of concern and fatigue loads adjusted by weigh station measurements or weigh-in-motion measurements at the bridge site. The fatigue provisions in "The LRFD," first released in 1994, refined "The Standard Specifications" in the expression of fatigue loading and fatigue resistance and the use of more realistic loading cycles. Although primarily intended for new bridge design, the LRFD fatigue provisions can also be used for existing bridge evaluation. The MBE, since its first release in 2008, has become the exclusive specification for fatigue evaluation of existing steel highway bridges. Literature is available on comparisons among these AASHTO fatigue specification documents (Chotickai and Bowman 2006; Zhou 2006a).

14.5.1.2 AREMA Fatigue Specifications for Railway Bridges

Fatigue specifications for the evaluation of railway bridges are similar to those of AASHTO in terms of classification of bridge structural details for their fatigue strengths and general philosophies of fatigue design and evaluation. Detailed provisions for fatigue design of new railway bridges and evaluation of existing railway bridges are available in Chapter 15, "Steel Structures," of MRE (AREMA 2012).

14.5.2 Load-Induced Fatigue and Distortion-Induced Fatigue

Fatigue damage on steel bridges has been traditionally categorized as either load induced or distortion induced. Load-induced fatigue is fatigue due to primary in-plane stresses in the steel plates that comprise bridge member cross sections. The stresses can be directly correlated with bridge live loads using conventional design theories and are typically calculated and checked in the fatigue design or evaluation process. The damage due to load-induced fatigue is steel cracking initiated from a source of stress concentration in a fatigue-susceptible detail. For example, a transverse crack that is initiated from a weld toe at the termination of a welded cover plate and propagated into the tension flange of a steel plate girder is a typical load-induced fatigue crack.

Distortion-induced fatigue is fatigue due to secondary stresses in the steel plates that comprise bridge member cross sections. These stresses, which are typically caused by out-of-plane forces, can only be calculated with very refined methods of analysis or measured by strain gages, far beyond the scope of a conventional bridge design or evaluation. To avoid distortion-induced cracks, detailing guidelines to minimize secondary stresses, such as connecting transverse connection plates for diaphragms and floor beams to both compression and tension flanges of girders, are provided in the LRFD design specifications (AASHTO 2012).

Figure 14.4 illustrates the mechanism of distortion-induced fatigue at an unsupported web gap between the upper end of a diaphragm connection plate and the girder top flange. Figure 14.5 shows an actual fatigue crack initiated from such a detail on a steel girder bridge. Previous research estimated that 90% of all fatigue cracking in steel bridges is the result of out-of-plane distortion at fatigue sensitive details (Connor and Fisher 2006). Common distortion-induced fatigue cracking sites include the web gap region at the end of vertical stiffeners or connection plates for floor beams, or in the web gap region of lateral gusset plates that intersect vertical connection plates. An early experimental examination of distortion-induced fatigue damage was reported in NCHRP Report 206 (Fisher et al. 1979) for various welded steel bridge details. A comprehensive study for development of design and retrofit criteria to minimize distortion-induced fatigue in steel bridges was reported in NCHRP Report 336 (Fisher et al. 1990). The latest research on distortion-induced fatigue was summarized in NCHRP Report 721 (Bowman et al. 2012) which included a literature review, a survey of bridge owners, as well as laboratory testing and general retrofit guidelines.

14.5.3 Classification of Steel Bridge Details for Fatigue Resistance

In accordance with the AASHTO bridge design specifications (AASHTO 2012), commonly used steel bridge details are classified into fatigue categories A, B, B′, C, C′, D, E, and E′ based on their fatigue characteristics. The so-called "*S-N* curves" define a lower bound fatigue resistance for each of the categories with

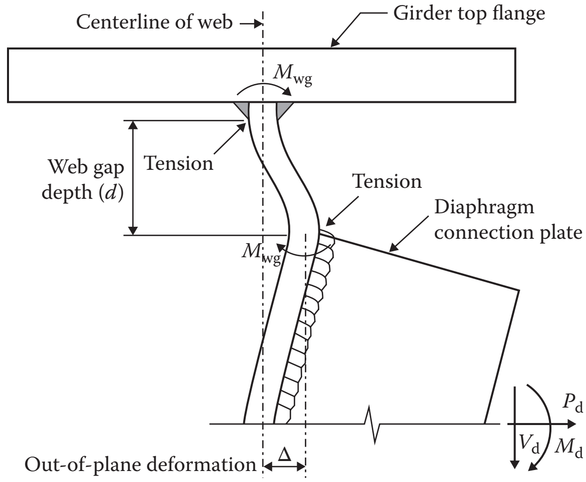

FIGURE 14.4 Illustration of distortion-induced fatigue mechanism (M_{wg} is bending moment in web; P_d, V_d, and M_d are horizontal axial force, vertical shear, and bending moment from diaphragm, respectively).

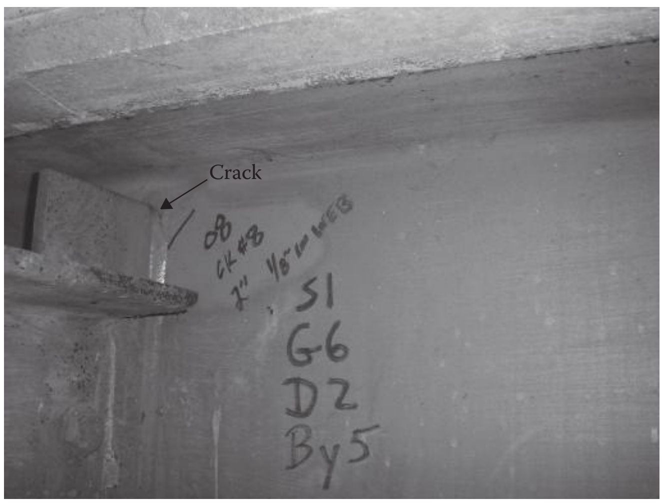

FIGURE 14.5 A distortion-induced fatigue crack on a steel girder bridge.

a 2.3% probability of failure (two standard deviations below the mean), where S is the stress range of a constant-amplitude cyclic loading and N is the number of cycles to a fatigue failure. The S-N curves were developed based on the laboratory testing of full-scale specimens primarily under constant-amplitude fatigue loading, although variable-amplitude fatigue tests were also conducted. Figure 14.6 shows the AASHTO S-N curves plotted in the log–log scale, in which the horizontal dashed lines define the constant-amplitude fatigue threshold (CAFT). No fatigue damage is assumed to occur if the stress range from a constant-amplitude loading is below the CAFT (values specified in the LRFD specifications for all fatigue categories).

The fatigue resistance for each fatigue category is defined by the following equation, as defined in AASHTO LRFD:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{1/3} \tag{14.8}$$

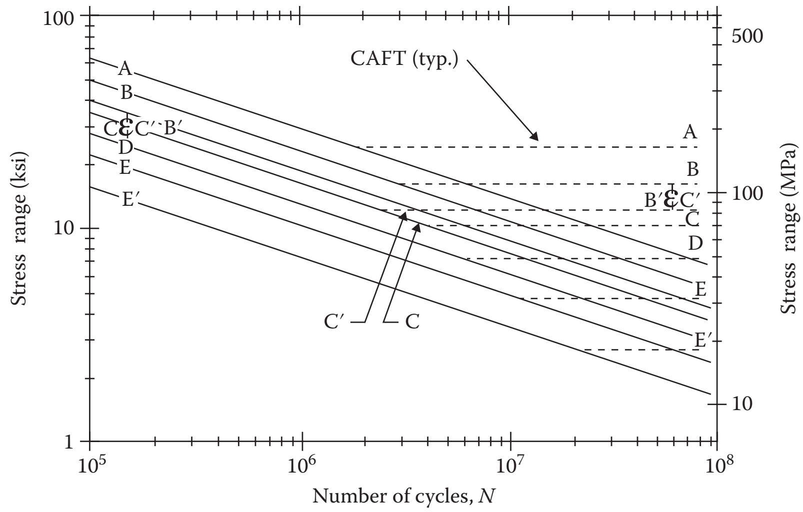

FIGURE 14.6 AASHTO fatigue resistance *S-N* curves for steel bridge details.

where $(\Delta F)_n$ is nominal fatigue resistance stress range, N is design fatigue life in the total number of stress cycles, and A is a constant defined in the LRFD for each of the fatigue categories.

Additional information is provided in MBE (AASHTO 2013) for the evaluation of riveted connections on existing steel bridges. The MBE suggests that the base metal at net sections of riveted connections of existing bridges be evaluated as category C fatigue detail instead of category D as specified in the LRFD for the design of new bridges to account for the internal redundancy of riveted members.

Recently completed NCHRP Project 12–81 (Bowman et al. 2012) provided further guidelines for the fatigue resistance of tack welds and riveted connections. Tack welds are common in old riveted steel structures, and their fatigue strength has not been well defined in previous specifications. It was suggested that tack welds of normal conditions be evaluated as a category C fatigue detail as opposed to category E for "base metal for intermittent fillet welds" as defined in previous AASHTO specifications. It was also suggested that for riveted members of poor physical condition, such as those with missing rivets or indications of punched holes, category D should be used.

14.5.4 Life Assessment of Load-Induced Fatigue

Bridge details are only considered prone to load-induced fatigue damage if they experience a net tensile stress under combined dead and live loads. For load-induced fatigue, the MBE (AASHTO 2013) contains provisions for two levels of fatigue evaluation: the infinite life check and the finite life calculations. Only bridge details that fail the infinite life check are subject to the more complex finite life evaluation.

It needs to be understood that AASHTO fatigue provisions are intended for the evaluation of bridge members that do not exhibit any fatigue damage. Once cracks are detected, the AASHTO fatigue provisions must be used with caution because most of the fatigue life of the detail has been exhausted and retrofitting measures should be initiated. Alternatively, a fracture mechanics approach can be used to evaluate the fatigue crack damage.

Perhaps the most important issue in bridge fatigue life assessment is to determine the variable-amplitude stress-range spectrum, or histogram, that the fatigue detail is exposed to and an effective stress range that can properly represent the entire histogram for equivalent fatigue damage. The following alternative methods have been provided in the MBE for estimating load-induced stress ranges for fatigue life assessment:

- Simplified analysis and LRFD fatigue truckloading
- Simplified analysis and truck weight from weigh-in-motion study

TABLE 14.6 AASHTO MBE Stress-Range Estimate Partial Load Factors

Fatigue Life Evaluation Methods	Analysis Partial Load Factor, R_{sa}	Truck-Weight Partial Load Factor, R_{st}	Stress Range Estimate Partial Load Factor, R_s^a
	For Evaluation or Minimum Fatigue Life		
Stress range by simplified analysis, and truck weight per LRFD Design Article 3.6.1.4	1.0	1.0	1.0
Stress range by simplified analysis, and truck weight estimated through weigh-in-motion study	1.0	0.95	0.95
Stress range by refined analysis, and truck weight per LRFD Design Article 3.6.1.4	0.95	1.0	0.95
Stress range by refined analysis, and truck weight by weigh-in-motion study	0.95	0.95	0.90
Stress range by field-measured strains	N/A	N/A	0.85
	For Mean Fatigue Life		
All methods	N/A	N/A	1.00

[a] In general, $R_s = R_{sa}R_{st}$.

- Refined analysis and LRFD fatigue truckloading
- Refined analysis and truck weight from weigh-in-motion study
- Field-measured strains

The effective stress range due to bridge loading $(\Delta f)_{\text{eff}}$ is defined as follows in the MBE:

$$(\Delta f)_{\text{eff}} = R_s \Delta f \tag{14.9}$$

where R_s is the stress-range estimate partial load factor, calculated as the product of analysis partial load factor R_{sa} and truck-weight partial load factor R_{st}, that is, $R_s = R_{sa}R_{st}$, as summarized in Table 14.6; and Δf is the field-measured effective stress range, or 75% of the calculated stress range due to the passage of the LRFD fatigue truck in a single lane or a fatigue truck determined by a truck survey or weigh-in-motion study.

The recently completed NCHRP Project 12–81 (Bowman et al. 2012) introduced a multiple presence factor for adjusting the calculated effective stress range based on AASHTO single-lane fatigue loading to account for the simultaneous presence of trucks in multiple lanes based on weigh-in-motion data.

Two sources of uncertainty are present in calculating the effective stress range, $(\Delta f)_{\text{eff}}$, for evaluating a fatigue detail: (1) the uncertainty associated with the analysis method, represented by the analysis partial load factor, R_{sa}, and (2) the uncertainty associated with the assumed effective fatigue truck weight, represented by the truck-weight partial load factor, R_{st}. As the uncertainty is reduced by the employment of more refined analysis methods or site-specific data, the increased certainty is reflected in the lowered partial load factors, as illustrated in Table 14.6.

14.5.4.1 Infinite Fatigue Life Check

The fatigue life of a fatigue-susceptible detail is infinite if all the stress ranges that the detail experiences throughout its service life are less than CAFT. The infinite fatigue life check criterion is expressed as follows in MBE:

$$(\Delta f)_{\max} \le (\Delta f)_{\text{TH}} \tag{14.10}$$

where $(\Delta f)_{max}$ is the maximum stress range expected at the fatigue-prone detail, which may be taken as $2.0(\Delta f)_{eff}$, and $(\Delta f)_{TH}$ is the CAFT given in LRFD.

The recently completed NCHRP Project 12–81 (Bowman et al. 2012) recommended that $(\Delta f)_{max}$ be taken as $2.0(\Delta f)_{eff}$ for calculated stress range due to a fatigue truck determined by a truck survey or weigh-in-motion study with $R_s = 1.0$ or the larger value of two times field-measured effective stress range or the field-measured maximum stress range, unless another suitable value is justified. An exception is for members directly subjected to wheel loads, such as the orthotropic deck, in which case $3.0(\Delta f)_{eff}$ may be used to account for larger variations.

14.5.4.2 Finite Fatigue Life Estimate

Three different levels of finite fatigue life estimate are defined in the MBE as follows:

- Minimum expected fatigue life (with a probability of failure of 2%, same as the design fatigue life)
- Evaluation fatigue life (with a probability of failure of 16%, less conservative than the design fatigue life)
- Mean fatigue life (with a probability of failure of 50%, representing the statistically most likely fatigue life)

The total finite fatigue life of a fatigue-prone detail (Y in years) can be estimated using the following formula, and the remaining fatigue life for an existing bridge may be computed by subtracting the current age from the total fatigue life:

$$Y = \frac{R_R A}{365n(\mathrm{ADTT})_{SL}\left[(\Delta f)_{eff}\right]^3} \tag{14.11}$$

where R_R is the resistance factor specified for evaluation, minimum, or mean fatigue life, as given in Table 14.7; A is the detail fatigue category constant provided in LRFD; n is the number of stress-range cycles per truck passage estimated per LRFD and MBE guidelines; and $(\mathrm{ADTT})_{SL}$ is the average number of trucks per day in a single lane averaged over the fatigue life estimated per LRFD and MBE guidelines.

The recently completed NCHRP Project 12–81 provided several refinements to finite fatigue life assessment, including (1) adding an Evaluation 2 fatigue life level, (2) providing a closed-form solution for total finite fatigue life using an estimated traffic growth rate and the present $(\mathrm{ADTT})_{SL}$, (3) introduction of fatigue serviceability index for measuring the performance of a structural detail with respect to its overall fatigue resistance, and (4) providing recommended actions for varying calculated values of the fatigue serviceability index (Bowman et al. 2012).

The general procedure of fatigue life assessment should be such that the initial infinite life check is made with the simplest, least refined stress-range estimate. If the detail passes this check, no further

TABLE 14.7 AASHTO MBE Resistance Factor for Evaluation, Minimum, or Mean Fatigue Life, R_R

Detail Category[a]	R_R		
	Evaluation Life	Minimum Life	Mean Life
A	1.7	1.0	2.8
B	1.4	1.0	2.0
B′	1.5	1.0	2.4
C	1.2	1.0	1.3
C′	1.2	1.0	1.3
D	1.3	1.0	1.6
E	1.3	1.0	1.6
E′	1.6	1.0	2.5

[a] From LRFD Design Table 6.6.1.2.3–1 and Figure 6.6.1.2.3–1.

refinement is required. However, if the initial analysis suggests that the detail does not have infinite fatigue life, a refined procedure may be considered to reestimate the stress range before the more complex procedures of finite life fatigue evaluation are initiated.

14.5.5 Fatigue Evaluation through Field Measurements

In bridge fatigue evaluation, a key task is to accurately determine live load-induced stress ranges at the fatigue details to be evaluated. Compared with analytical methods, field strain measurement is most accurate since no assumptions need to be made for uncertainties in load distribution such as unintended composite action between structural components, contribution of nonstructural members, stiffness of various connections, and behavior of concrete decks in tension. The actual strain histories experienced by bridge components are directly measured by strain gages at the areas of concern. The effects of varying vehicle weights and their random combinations in multiple lanes are also reflected in the measured strains. The measurement results are then processed and expressed in terms of stress-range histograms using some cycle-counting algorithms such as the rain-flow method or the peak-to-peak method. These algorithms detect the peaks and valleys of strain history and determine cycle counts for all levels of measured stress ranges. The stress-range histogram presents the occurrence of stress cycles in terms of the number of cycles for each stress-range magnitude captured during the measurement period.

Since fatigue strength *S-N* curves were developed primarily under constant-amplitude cyclic loading, an effective stress range needs to be determined to represent the actual variable-amplitude cyclic loading for equivalent fatigue damage in a bridge detail. For steel structures, the root-mean-cube stress range calculated from a variable-amplitude stress-range histogram has been found to produce the best results for this purpose (Miner 1945, AASHTO 1990), as illustrated in Figure 14.7. Note that S_r and S_{re} have been used interchangeably with Δf and Δf_{eff} for stress range and effective stress range, respectively, in the literature under the subject of fatigue:

$$\Delta f_{eff} = [\Sigma \gamma_i \Delta f_i^3]^{1/3} \tag{14.12}$$

where Δf_{eff} is effective stress range of a variable-amplitude stress-range histogram; Δf_i is the *i*th stress range in the stress-range histogram; γ_i is n_i/N, that is, fraction of occurrence of stress range Δf_i in the histogram; n_i is the number of occurrences of stress range Δf_i; and N is the total number of occurrences of all stress cycles in the histogram.

The stress-range histogram illustrated in Figure 14.7 represents a typical form from strain measurements of highway bridge components. It consists of a high number of low-magnitude stress-range cycles

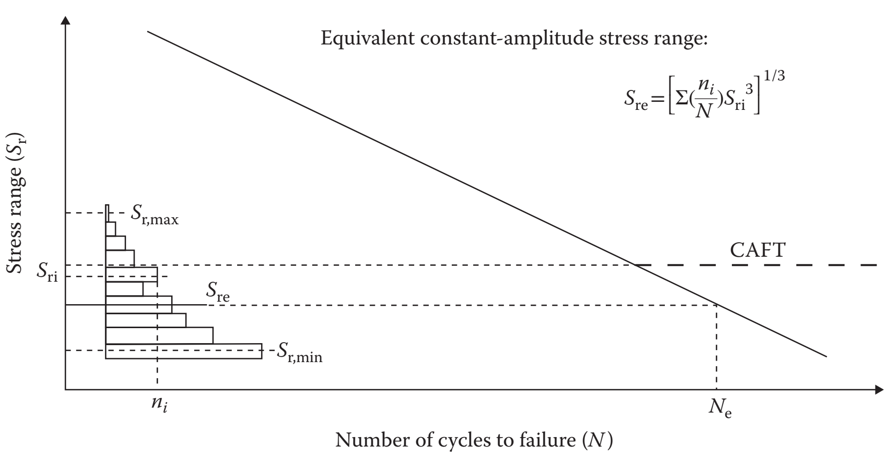

FIGURE 14.7 Fatigue analysis using *S-N* curve and measured stress-range histogram.

and a low number of high-magnitude stress-range cycles. The stress cycles of the lowest magnitudes are usually due to light vehicles and secondary vibrations, or even electrostatic or elctromagnetic noises if the strain gage signals are not filtered.

If the entire stress-range histogram is below the CAFT, that is, $S_{r,max} = \Delta f_{max} < \text{CAFT}$, the loading should not cause fatigue cracking at the detail and infinite fatigue life may be assumed, as specified in AASHTO MBE. For load-induced fatigue, this situation happens primarily to the higher fatigue resistance categories (C and above) that have relatively high CAFTs. The ratio CAFT/Δf_{max} provides a measurement of the factor of safety for infinite fatigue life. The acceptance level of CAFT/Δf_{max} for a bridge should be determined based on several case-specific factors (Zhou 2006b): (1) possibility of missing heavy vehicles during the measurement period (the longer the measurement, the lower the chance of missing heavy vehicles), (2) errors in strain measurement due to local geometry and shear lag effects, (3) physical condition of the detail due to corrosion/deterioration and quality of fabrication/welding workmanship, and (4) redundancy and importance of the bridge member that contains the detail in question.

The fatigue life of a detail will be finite as long as some stress ranges exceed the CAFT of the detail. In some cases, only a small fraction of the stress-range histogram is higher than CAFT, whereas the majority of the measured stress cycles are lower. For load-induced fatigue, this situation may happen to the lower fatigue resistance categories (D and below) that have relatively low CAFTs. In such a situation, including all the stress cycles in the as-measured histogram lowers the value of the calculated effective stress range and thus results in an overestimated fatigue life. Therefore, a lower truncation stress range, or cutoff, needs to be established for reducing the measured stress-range histograms. In other words, only the stress ranges equal to or higher than the truncation stress range are used in Equation 14.12 for calculating the effective stress range. The higher the truncation stress range, the higher the calculated effective stress range.

The magnitude of truncation stress range should be a fraction of CAFT of the fatigue detail. Based on research results of variable-amplitude fatigue in the United States and Europe, using 0.5CAFT as the trucation stress range has been considered reasonable yet conservative. That is, all stress ranges lower than 0.5CAFT are excluded in the calculation of effective stress range S_{re}. Several case-specific factors should also be considered in determining the truncation stress range (Zhou 2006b): (1) level of ambient electrostatic and electromagnetic noises if signals are not filtered, (2) stress cycles due to light vehicles based on the results of a calibration test that correlates the strain responses with a control vehicle of known weight, and (3) traffic information such as average daily traffic and ADTT counts.

For cases of finite fatigue life, the total fatigue life in terms of effective stress cycles, N_e, can be calculated from the *S-N* curve once an effective stress range, $S_{re} = \Delta f_{eff}$, is determined from a measured histogram (Figure 14.7). The bridge fatigue life in years can then be determined based on a correlation between the effective stress cycles, N_e, and the calendar time it represents. The remaining fatigue life is finally obtained by subtracting the current age from the total fatigue life with the consideration of loading history and predicted growth rates of traffic volume and weight.

Prior to field strain measurement, a structural analysis using two-dimensional or 3-D computer models should be performed to identify fatigue-critical members. The selection of members for field instrumentation is usually based on results of the stress analysis and the member type. Additionally, careful thought should be given to physical condition (such as section loss due to corrosion) and secondary stresses that are not considered in the analysis, such as frozen pins, dysfunctional expansion bearings, and bending stresses in axial members. The selection of strain gage locations should ensure true representation of the nominal stress range defined for the specific fatigue detail. Effects of local geometry and shear lag on stress distribution at the fatigue detail should be considered and the gage location or the measured strain adjusted as necessary.

A calibration test should be conducted with a vehicle, or multiple vehicles, of known weight to correlate local strain responses with the magnitudes and locations of the test loads applied. For measurement of stress-range histograms under normal traffic, the time period should be at least 7 consecutive days (1 week) to represent the basic unit of traffic pattern repetition. Longer measurement periods produce better representative stress-range histograms and reduce the possibility of missing heavy vehicles.

14.5.6 General Procedure for Life Assessment of Load-Induced Fatigue

A general procedure for assessing the fatigue life of bridge details due to load-induced fatigue is summarized as follows:

1. Identify fatigue details on the bridge, and determine their fatigue resistance categories per AASHTO LRFD fatigue provisions. Perform a structural analysis using a simplified analysis method to find those that experience a net tensile stress under a combined dead load and a maximum fatigue load twice the AASHTO fatigue truck. Fatigue evaluation needs to be performed only to these fatigue-prone details.
2. Check for the MBE infinite life requirement by comparing the maximum stress range, $(\Delta f)_{max}$, with CAFT for all fatigue-prone details. Fatigue evaluation is complete if $(\Delta f)_{max} \leq$ CAFT, that is, infinite life is expected. Otherwise, use a refined procedure to reestimate $(\Delta f)_{max}$, including 3-D finite element models and/or site-specific truck weight data through a weigh-in-motion study.
3. If the refined $(\Delta f)_{max}$ is still higher than CAFT, calculate the remaining finite fatigue life in years per the MBE procedures for estimating finite fatigue life. Fatigue evaluation is complete if the calculated remaining life satisfies the expected service need of the bridge.
4. If the remaining fatigue life turns out to be insufficient for expected service, consider field strain measurement for the possibility of a lower effective stress range due to reasons such as unintended composite action, participation of secondary members, different load distribution than that assumed in the analysis, and different truck weights and travel patterns than those used in the analysis.
5. Develop an instrumentation plan including strain gage locations, sampling rates, measurement time period, power supply, and so on. Install strain gages and connect them to data acquisition systems. Perform a calibration test with a vehicle, or several vehicles, of known weight to cross each lane of the bridge; and establish baseline correlations between test loads and their stress responses at the fatigue details of concern.
6. Record data continuously (24/7), and generate stress-range histograms from each strain gage for a time period that is considered sufficient.
7. Check infinite fatigue life requirement: $(\Delta f)_{max} \leq$ CAFT. If this requirement is satisfied, the ratio CAFT/$(\Delta f)_{max}$ measures the reliability of infinite fatigue life considering the measurement period, nominal stress, physical condition, and member redundancy.
8. If the infinite fatigue life requirement is not satisfied, estimate the remaining life in years following MBE procedures for estimating finite fatigue life using a measured stress-range histogram. Calculate the effective stress range as $(\Delta f)_{eff} = [(n_i/N)\ \Delta f_i^3]^{1/3}$ from the measured histogram by excluding all stress ranges less than 0.5CAFT. Also, consider loading history and future growth for $(\Delta f)_{eff}$.
9. Summarize procedure and fatigue evaluation results in a report. Tabulate $(\Delta f)_{max}$, $(\Delta f)_{eff}$, CAFT, and Y_{remain} for each fatigue-prone detail as applicable.

14.5.7 Evaluation and Retrofit of Distortion-Induced Fatigue Cracks

For welded steel plate girders constructed prior to the late 1970s, a common practice was to avoid transverse welds on the tension flange. The stiffener/connection plate for the diaphragm or cross frame is often welded to the compression flange but not connected to the tension flange of the girder. As a result, an unsupported web gap forms between the end of the connection plate and the tension flange of the girder. Under the live load-induced forces in the diaphragms or cross frames, the unsupported web gap is subject to out-of-plane distortion, as depicted in Figures 14.4 and 14.5. The maximum tensile stresses caused by such distortion occur at the toes of the horizontal web-to-flange welds and the vertical web-to-connection plate welds. These stresses cause horizontal cracks in the web or the web-to-flange welds and horizontal or vertical cracks at the end of the web-to-connection plate welds. As crack lengths grow under cyclic loading, the directions of the cracks also change to being perpendicular to the principal tensile stress in the local area, which varies during the progression of crack length. If not discovered timely and treated properly, distortion induced cracks may cause large fractures in steel girders (Zhou and Biegalski 2010).

The primary driving force for distortion-induced fatigue cracks is the lateral forces from the diaphragm and cross-frame members due to differential deflections of adjacent girders under live loads. Distortion-induced fatigue only occurs at locations where the connection plate is not positively connected to the girder flange (with an unsupported web gap). The occurrence and the severity of distortion-induced fatigue depend on many factors including the magnitudes of the lateral forces on both sides of the girder web, length of the unsupported web gap, thickness of the web plate, and quality of welding.

There are two general repair schemes for distortion-induced fatigue problems: one is to make the detail more flexible by increasing the depth of the web gap, which is called the "softening approach." For a given out-of-plane deformation, a greater web gap depth should result in lower local stresses at the ends. However, this scheme works only if the increased flexibility does not cause greater out-of-plane deformation of the unsupported web. The other repair scheme is to make the web gap detail a stiff connection by providing a positive connection between the girder flange and the stiffener/connection plate, which is called the "stiffening approach." The key for success of this repair scheme is to ensure the strength, and more importantly the stiffness, of the new connection since previous research has indicated that a lateral movement of a few thousandths of an inch can induce local stresses of up to 20 ksi. Either approach can work if configured and executed properly.

Figures 14.8 and 14.9 depict examples of the results of a softening repair. The original design was such that cross-frame connection plates were welded to the compression flange of the exterior girders but were just "close fit" (without welding) to both flanges of the interior girders. Fatigue cracks occurred in web gap areas near the top flange of the interior girders where the connection plate was not welded to the top flange. Repairs were made to drill holes at the crack tips and to cut short the connection plate to create a large web gap for reducing the local stresses. A portion of the connected cross-frame diagonal was also removed and then reconnected to the altered connection plate (Figure 14.9). Some years later, fatigue cracks reinitiated and propagated beyond the initially drilled holes. As shown in Figure 14.8, the big hole in the center was drilled initially to remove the original upper end of the connection plate-to-web weld.

FIGURE 14.8 A repaired web gap detail with a cut-short connection plate and drilled holes.

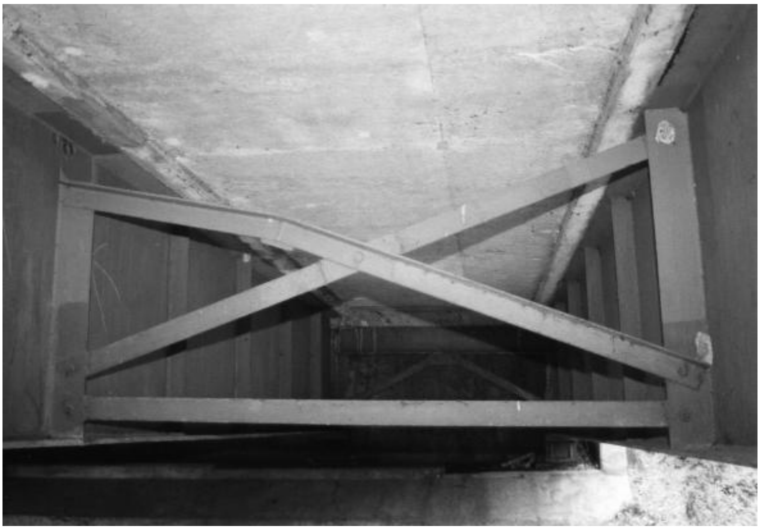

FIGURE 14.9 Modified cross frame between adjacent girders for softening repair.

The two small holes were also drilled at the same time to remove the tips of the cracks from the original web gap detail. The two large holes at the sides were drilled years later after the propagation of the horizontal crack along the toe of the flange-to-web weld. Field strain measurements indicated cyclic vertical stresses up to 15 ksi near the top of the web caused by lateral cross-frame member forces under live loads. The relatively high stresses are a result of reduced stiffness by the drilled holes and the cut-short connection plates in the web gap area. Stresses of such magnitude are indications of possible continuous cracking from the weld toe.

For repairs of distortion-induced cracks using the stiffening approach, it is critical to make the connection very stiff for the lateral forces from the diaphragms, cross frames, or floor beams. The magnitudes of these forces are often very difficult to accurately quantify even using comprehensive computer models or field strain measurements. Repairs based on underestimated lateral forces are likely to be ineffective in stopping cracks from growing. When high-strength bolts are used for repairs, a sufficient number must be used with slip-critical connections. Figure 14.10 shows an unsuccessful repair where Huck bolts were installed from the underside without access to the topside of the girder top flange. Huck bolts provide ease of construction by installing the bolts without removing the concrete above the girder flange. However, the nature of such bolts and the surface condition did not warrant a nonslip, stiff connection, although the bearing capacity may satisfy the strength requirement. At this location (Figure 14.10) the head of both Huck bolts fractured off, and at a few other locations across the four-span bridge distortion-induced fatigue cracks continued propagating a few years after the repair. Subsequently welded repairs were installed after field strain measurements indicated low tensile strains at locations of field welding. Figure 14.11 shows the completed field-welded repair.

In summary, distortion-induced fatigue cracks in bridge connections are often the results of localized stresses or distortions resulting from member interactions of global structural behavior that was not properly considered in the original design. Successful repairs for distortion-induced fatigue cracks require a good understanding of the structural behavior to accurately identify the driving force for localized stress concentration or distortion. A thorough understanding of structural behavior can be obtained from examining the global structural behavior and the interaction between connected components. A combination of 3-D computer models and field strain measurements provides good tools for such an analysis.

FIGURE 14.10 An unsuccessful repair of distortion-induced fatigue using Huck bolts.

FIGURE 14.11 Repair of distortion-induced fatigue using field welding.

14.6 Fracture Criticalness and Structural Redundancy

14.6.1 Steel Fracture and Fracture Control

Steel fracture is a process of rupture under tension associated with the creation of new surfaces. The process of fracture is usually brittle in nature and happens very rapidly. There are two general types of steel fracture: one is caused when load-induced average stress across a member section exceeds the tensile strength of the steel ($S > F_u$), and the other is caused when the stress intensity factor in fracture mechanics exceeds the fracture toughness of the steel ($K_I \geq K_{Ic}$) due to the presence of a crack while the average stress in the steel is well below the yield strength ($S < F_y$).

Steel fracture can happen under constant loading, as the result of fatigue under cyclic loading or a combination of them. Fatigue-induced fracture occurs when the size of a flaw, or crack, reaches a critical size for the material's facture toughness. In this case, the magnitude of total stress is important for fracture as opposed for the cyclic stress range is for fatigue.

For fracture control of steel bridges in design, AASHTO LRFD (AASHTO 2012) stipulates: "Except as specified herein, all primary longitudinal superstructure components and connections sustaining tensile stress due to Strength Load Combination I, as specified in Table 3.4.1–1, and transverse floor beams subject to such stress, shall require mandatory Charpy V-notch testing." LRFD specifications provide minimum values for Charpy V-notch impact energy requirements for multiple types of steel in varying thicknesses for three different temperature zones in Article 6.6.2, Fracture.

14.6.2 Fracture-Critical Members in Steel Bridges

AASHTO LRFD specifications define a fracture-critical member (FCM) as a "component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function." It should be noted that FCM can refer to a component such as a flange of a girder and does not necessarily include the whole "member." Approximately 11% of steel bridges in the United States have FCMs. Most of these (83%) are two-girder bridges and two-line trusses, and 43% of the FCMs are built-up riveted members (Connor et al. 2005). Since 1988, the FHWA's National Bridge Inspection Standards have mandated "hands-on" fracture-critical inspection to bridges that contain FCMs. Typically, FCMs refer to steel members only.

14.6.3 Evaluation of Bridge Structural Redundancy

MBE (AASHTO 2013) defines bridge redundancy as "the capability of a bridge structural system to carry loads after damage or the failure of one or more of its members." FCMs are nonredundant. But nonredundancy is a broader term than FCM since it also includes failure modes other than fracture such as buckling, corrosion, or failures of non-steel members or substructure elements.

In AASHTO MBE, structural redundancy is reflected in the system factor, φ_s, that reduces the capacity of members in nonredundant systems. The system factor was calibrated so that nonredundant systems are rated more conservatively at approximately the same level of reliability associated with new bridges designed by LRFD specifications.

Redundancy is related to system behavior rather than individual component behavior. Redundancy is often discussed in terms of three types (Connor et al. 2005):

1. Internal redundancy, also called member redundancy, can occur when a nonwelded member comprises multiple elements, and a fracture that is formed in one element cannot propagate directly to the adjacent elements.
2. Structural redundancy is external static indeterminacy and can occur in a continuous girder or truss with two or more spans.
3. Load-path redundancy is internal static indeterminacy resulting from having three or more girders or redundant truss members. One can argue (and analytically show) that transverse members such as diaphragms between girders can also provide load-path redundancy.

Ultimately, it is the target level of reliability that bridge evaluation engineers should strive to achieve, rather than focusing exclusively on redundancy. Redundancy has a major impact on the risk of collapse, and this impact is accounted for appropriately for all types of structures in both AASHTO MBE and LRFD.

The capacity of damaged superstructures (with the FCM "damaged" or removed from the analysis) may be predicted using a refined 3-D analysis. In AASHTO LRFD Article 6.6.2—Fracture—the commentary states: "The criteria for a refined analysis used to demonstrate that part of a structure is not fracture critical have not yet been codified. Therefore, the loading cases to be studied, location of potential cracks, degree to which the dynamic effects associated with a fracture are included in the analysis, and fineness of models and choice of element type should all be agreed upon by the Owner and the Engineer. The ability of a particular software product to adequately capture the complexity of the problem should also be considered and the choice of software should be mutually agreed upon by the Owner and the Engineer. Relief from the full factored loads associated with the Strength I Load Combination of Table 3.4.1–1 should be considered, as should the number of loaded design lanes versus the number of striped traffic lanes."

NCHRP Report 406, *Redundancy in Highway Bridge Superstructures*, provides guidelines for estimating the residual capacity of a damaged superstructure (Ghosn and Moses 1998).

14.7 Summary

This chapter discusses background, criteria, and detailed procedures for the evaluation of highway and railway steel bridges. The subjects of structural evaluation include load rating, remaining life assessment for load-induced fatigue, evaluation and retrofit of distortion-induced fatigue, as well as fracture and redundancy evaluation. The primary code specifications referenced are AASHTO MBE (AASHTO 2013), AASHTO *LRFD Bridge Design Specifications* (AASHTO 2012), and AREMA MRE (AREMA 2012).

References

AASHO. 1931. *Standard Specifications for Highway Bridges and Incidental Structures*. American Association of State Highway Officials, Washington, D.C.

AASHO. 1941. *Standard Specifications for Highway Bridges*. Third Edition, American Association of State Highway Officials, Washington, D.C.

AASHTO. 1990. *Guide Specifications for Fatigue Evaluation of Existing Steel Bridges*, interim specifications in 1993 and 1995, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. 1994. *Manual for Condition Evaluation of Bridges*, Second Edition. American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. 2002. *Standard Specifications for Highway Bridges*, Seventeenth Edition. American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. 2003. *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges.* American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. 2012. *LRFD Bridge Design Specifications*, Customary U.S. Unit, 2012. American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. 2013. *The Manual for Bridge Evaluation*, Second Edition, with 2011 and 2013 Interim Revisions. American Association of State Highway and Transportation Officials, Washington, D.C.

AREMA. 2012. *Manual for Railway Engineering*. American Railway Engineering and Maintenance-of-way Association, Lanham, MD.

Bowman, M.D., Fu, G., Zhou, Y.E., Connor, R.J., and Godbole, A.A. 2012. *Fatigue Evaluation of Steel Bridges*, NCHRP Report 721. Transportation Research Board, Washington, D.C.

Chotickai, P. and Bowman, M.D. 2006. Comparative Study of Fatigue Provisions for the AASHTO Fatigue Guide Specifications and LRFR Manual for Evaluation, *Journal of Bridge Engineering*, ASCE, Vol. 11, No. 5 (September/October): 655–660.

Connor, R.J., Dexter, R., and Mahmoud, H. 2005. *Inspection and Management of Bridges with Fracture-Critical Details, A Synthesis of Highway Practice*, NCHRP Synthesis 354. Transportation Research Board, Washington, D.C.

Connor, R.J. and Fisher, J.W. 2006. Identifying Effective and Ineffective Retrofits for Distortion Induced Fatigue Cracking in Steel Bridges using Field Instrumentation. *Journal of Bridge Engineering*, ASCE, Vol.11, No. 6 (November/December): 745–752.

Fisher, J.W., Hausammann, H., Sullivan, M.D., and Pense, A.W. 1979. *Detection and Repair of Fatigue Damage in Welded Highway Bridges*, NCHRP Report 206, Transportation Research Board, Washington, D.C.

Fisher, J.W., Jin, J., Wagner, D.C., and Yen, B.T. 1990. *Distortion-Induced Fatigue Cracking in Steel Bridges*, NCHRP Report 336, Transportation Research Board, Washington, D.C.

Ghosn, M. and Moses, F. 1998. *Redundancy in Highway Bridge Superstructures*, NCHRP Report 406, Transportation Research Board, Washington, D.C.

Miner, M.A. 1945. Cumulative Damage in Fatigue. *Journal of Applied Mechanics,* 12(1), 159–164.

Moses, F., Schilling, C.G., and Raju, K.S. 1987. *Fatigue Evaluation Procedures for Steel Bridges*, NCHRP Report No. 299. Transportation Research Board, Washington, D.C.

NCHRP. 1998. *Manual for Bridge Rating through Load Testing*, NCHRP Research Results Digest No. 234. Transportation Research Board, Washington, D.C.

USDOT FHWA. 2009. *Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges*. U.S. Department of Transportation, Federal Highway Administration, Publication No. FHWA-IF-09-014. http://bridges.transportation.org/Documents/FHWA-IF-09-014LoadRatingGuidanceandExamplesforGussetsFebruary2009rev3.pdf (Accessed September 3, 2013).

Zhou, Y.E. 2006a. Bridge Fatigue Evaluation per AASHTO Specifications, First International Conference on Fatigue and Fracture in the Infrastructure, Philadelphia, PA.

Zhou, Y.E. 2006b. Assessment of Bridge Remaining Fatigue Life through Field Strain Measurement. *Journal of Bridge Engineering*, ASCE, Vol. 11, No. 6 (November/December): 737–744.

Zhou, Y.E. and Biegalski A.E. 2010. Investigation of Large Web Fractures of a Welded Steel Plate Girder Bridge. *Journal of Bridge Engineering*, ASCE, Vol. 15, No. 4 (July/August): 373–383.

15

Concrete Bridge Evaluation and Rating

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15.1 Introduction

Once a bridge is constructed, it becomes the property of the owner or agency. The bridge owner is responsible for the maintenance of the bridge and ensuring the safety of the traveling public. Before the 1960s, little emphasis was given to inspection and maintenance of bridges in the United States. After the 1967 tragic collapse of the Silver Bridge at Point Pleasant in West Virginia, national interest in the inspection and maintenance arose considerably. The U.S. Congress passed the "Federal Highway Act of 1968," which resulted in the establishment of the National Bridge Inspection Standard (NBIS). The NBIS sets the national policy regarding the bridge inspection procedure, inspection frequency, inspector qualifications, reporting format, and rating procedures.

In the United States, since the highway bridges are designed for the American Association of State and Highway Transportation Officials (AASHTO) design vehicles, most U.S. engineers tend to believe that the bridge will have adequate capacity to handle the actual present traffic. This is generally true provided that the bridge was constructed and maintained as shown in the design plan. However, changes in few details during the construction phase, failure to attain the recommended concrete strength, unexpected settlements of foundation after construction, and unforeseen damages to a member could lower the capacity of the bridge. In addition, older bridges might have been designed using different design code and for lighter vehicles than that is used at present. Current AASHTO Specifications incorporate the latest research findings on bridge structures and current traffic conditions. As a result, it is prudent to evaluate and rate these bridges using most up-to-date specifications. By doing this evaluation or rating, bridge owners can ensure the public safety. NBIS requires that whenever the structural condition of a bridge changes or any additional loading is placed on the bridge, the bridge load-carrying capacity should be reestablished.

Sometimes, an industry would like to transport their heavy machinery from one location to another location. Similarly, in some cases, contractors would like to transport larger earth moving equipment, cranes, and other construction vehicles on the bridges. These vehicles would be much heavier than the design vehicles and thus the bridge owner may need to determine the current live load-carrying capacity of the bridge.

The evaluation or rating of the existing bridges is a continuous activity of the agency to ensure the safety of the public. The evaluation provides necessary information to repair, rehabilitate, post, close, or replace the existing bridge.

Chapter 13 discusses the importance of bridge inspection and related activities to ensure safety of traveling public and trucks. Chapter 14 presents the basics of bridge rating and posting. The current chapter describes the concrete bridge rating procedures and provides several illustrative examples.

15.2 Concrete Bridge Rating

15.2.1 Rating Principles

In general, the resistance of a structural member (R) should be greater than the demand (Q) as follows:

$$R \geq Q_{\mathrm{DL}} + Q_{\mathrm{LL}} + \sum_{k} Q_{\mathrm{k}} \tag{15.1}$$

where Q_{DL} is the effect of dead load, Q_{LL} is the effect of vehicular live load, and Q_{k} is the effect of load k.

Equation 15.1 applies to design and as well as evaluation. During bridge design phase, design engineers attempt to establish capacity (R) that meets Equation 15.1. During the bridge evaluation process, on the contrary, maximum allowable vehicular live load that can be placed on the bridge is determined. After rearranging Equation 15.1, the maximum allowable vehicular live load will be

$$Q_{\mathrm{LL}} \leq R - \left(Q_{\mathrm{DL}} + \sum_{k} Q_{\mathrm{k}} \right) \tag{15.2}$$

Maintenance engineers always determine whether a fully loaded vehicle (rating vehicle) can be allowed on the bridge and if not what portion of the rating vehicle could be allowed on the bridge. The portion of the rating vehicle that can be safely placed on the bridge will be given by the ratio between the available capacity for live load effect and the effect of the rating vehicle. This ratio is called rating factor (RF). Maintenance engineers assume that the weight carried by each axle will be adjusted by the same proportion of RF.

$$\mathrm{RF} = \frac{\text{Available capacity for the live load effect}}{\text{Rating vehicle load demand}} = \frac{R - \left(Q_{\mathrm{DL}} + \sum_{k} Q_{\mathrm{k}} \right)}{Q_{\mathrm{LL}}} \tag{15.3}$$

When the rating factor equals or exceeds unity, the bridge is capable of carrying the rating vehicle. On the contrary when the rating factor is less than unity, the bridge may be overstressed while carrying the rating vehicle.

15.2.2 Rating Philosophies

During the structural evaluation process, the location and type of critical failure modes are first identified. Equation 15.3 is then solved for each of these potential failures. Although the concept of evaluation is the same, the mathematical relationship of this basic equation for allowable stress rating (ASR), load factor rating (LFR), and load and resistance factor rating (LRFR) differs. Since the resistances and load effects can never be established with certainty, engineers use safety factors to give adequate assurance against failure. ASR includes safety factors in the form of allowable stresses of the material. LFR considers the safety factors in the form of load factors to account for the uncertainty of the loadings and resistance factors to account for the uncertainty of structural response. LRFR also treats safety factors in the form of load and resistance factors; however, these load and resistance factors, unlike in the LFR, are based on the probability of the loadings and resistances and based on set target reliability of failure.

For ASR, the rating factor expression, Equation 15.3, can be written as

$$\mathrm{RF} = \frac{R - \left(\sum_{n=1}^{N} D_\mathrm{n} + \sum_{k=1}^{K} L_\mathrm{k}(1+I) \right)}{\mathrm{LL}(1+I)} \tag{15.4}$$

For LFR, the rating factor expression, Equation 15.3 can be written as

$$\mathrm{RF} = \frac{\varphi R_\mathrm{n} - \sum_{n=1}^{N} \gamma_{D_\mathrm{n}} D_\mathrm{n} - \sum_{k=1}^{K} \gamma_{L_\mathrm{k}} L_\mathrm{k}(1+I)}{\gamma_{\mathrm{LL}} \mathrm{LL}(1+I)} \tag{15.5}$$

For LRFR, the rating factor expression, Equation 15.3, can be written as

$$\mathrm{RF} = \frac{\varphi R_\mathrm{n} - \sum_{n=1}^{N} \gamma_{D_\mathrm{n}} D_\mathrm{n} - \sum_{k=1}^{K} \gamma_{L_\mathrm{k}} L_\mathrm{k}(1+I)}{\gamma_{\mathrm{LL}} \mathrm{LL}(1+I)} \tag{15.6}$$

where R is the allowable stress of the member, φR_n is nominal resistance, D_n represents the effect of dead loads and permanent loads, L_k is the live load effect for load k other than the rating vehicle, LL is nominal live load effect of the rating vehicle, I is the impact factor for the live load effect, γ_{D_n} is dead load factor, and γ_{L_k} and γ_{LL} are live load factors. The load factor for dead loads and live loads could vary with the load type. For example, the dead load factor for self-weight and wearing surface within LRFR method is 1.25 and 1.50, respectively.

To use Equation 15.4 through 15.6 in determining the rating factors, one needs to estimate the effects of individual live load vehicles. The effect of individual live load vehicles on structural member could only be obtained by analyzing the bridge using a three-dimensional analysis. Thus, obtaining the rating factor using the preceding expressions is very difficult and time consuming.

Furthermore, L_k in Equation 15.4 through 15.6 should be based on the vehicles that are on the structure while the rating vehicle is traveling on the bridge. Type of vehicles and weight distributions of the vehicles will have significant impact on the rating. However, it is extremely difficult to establish these values. In recent years, there were extensive studies performed by various state agencies using Weigh-in-Motion records to establish the average configuration and load effect of trucks traveling beside the rating vehicles. These studies recognized the complexity of highway loading and recommended a simplified approach to evaluating the bridges. Recommended simplified approach is adapted within LRFR method and incorporated within The Manual for Bridge Evaluation (MBE) (AASHTO 2013).

Two simplified approaches are recommended in MBE Specification. First one is to assume that similar rating vehicles will occupy all the possible lanes to produce the maximum effect on the structure. This assumption allows engineers to use the AASHTO live load distribution factor (LLDF) approach to estimate the live load demand and eliminate the need for the three-dimensional analysis. This methodology has been used by engineers in the past within ASR and LFR methods. Second approach is for a scenario where effect of rating vehicle is significantly larger than that of the trucks traveling on the bridge. MBE adapted a methodology in which the effect of the lighter vehicle traveling beside the rating vehicle is ignored, but accounted for by including little higher live load factor on the rating vehicle. This method is available for evaluating bridges within LRFR method only.

The simplified rating factor equations based on similar vehicles in all lanes then become

$$\text{For ASR,} \qquad \mathrm{RF} = \frac{R - \sum_{n=1}^{N} D_\mathrm{n}}{\mathrm{LL}(1+I)} \tag{15.7}$$

$$\text{For LFR,} \qquad \mathrm{RF} = \frac{\varphi R_\mathrm{n} - \sum_{n=1}^{N} \gamma_{D_\mathrm{n}} D_\mathrm{n}}{\gamma_\mathrm{LL}\mathrm{LL}(1+I)} \tag{15.8}$$

$$\text{For LRFR,} \qquad \mathrm{RF} = \frac{\varphi R_\mathrm{n} - \sum_{n=1}^{N} \gamma_{D_\mathrm{n}} D_\mathrm{n}}{\gamma_\mathrm{LL}\mathrm{LL}(1+I)} \tag{15.9}$$

In the derivation of Equations 15.7 through 15.9, it is assumed that the resistance of the member is independent of the loads. Few exceptions to this assumption are beam column members and beams with high moment and shear. In a beam column member, axial capacity and moment capacity are functions of applied moment and applied axial load, respectively. Figure 15.1 shows the capacity of a beam column member (P-M diagram) at a section. From this figure, it can be seen that the member with axial load of P_o and P_c1 will have moment capacity of M_c0 and M_c1, respectively. And the member with moment demand of M_o and M_c1 will have axial capacity of P_c0 and P_c1, respectively. Thus, as the live load forces from the rating vehicle is increased from (P_o, M_o) to (P_c1, M_c1), the axial capacity of the member drops from P_c0 to P_c1 and moment capacity of the member increases from M_c0 to M_c1. In other words, the numerator of the preceding equations (available live load capacity) will change as the live load changes. Thus, the available live load capacity will be a function of live load.

This example illustrates that whenever the capacity varies with the demand, iterative process needs to be utilized to establish the rating factor.

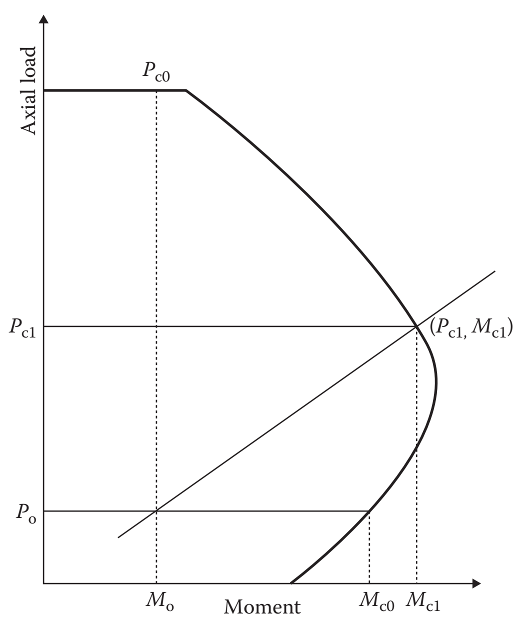


FIGURE 15.1 P-M diagram of a beam column.

15.2.3 Level of Ratings

In general, there are two levels of rating available for highway bridges: inventory and operating. The load that can be safely carried by a bridge for indefinite period is called an inventory rating. This level of rating typically reflects the "design" level and is achieved by setting higher safety margin. Within ASR method, allowable stress of materials was set to a lower value. Within LFR, higher live load factors (1.3 × beta factor 1.67 = 2.167) are employed, and within the LRFR method, the live load and resistance factors were set so that a reliability index of 3.5 is achieved.

The rating that reflects the absolute maximum permissible load that can be safely carried by the bridge is called an operating rating. Within ASR method, allowable stress of materials is increased, but kept below the yield stress. Within LFR live load, beta factor is reduced from 1.67 to 1.0 that yields a live load factor of 1.3, and within LRFR, live load factor is adjusted so that a reliability index of 2.5 is achieved.

The life of a bridge depends not only on its ultimate strength but also on the frequency of loading and unloading and flexibility of the bridge. Frequent loading and unloading may affect the fatigue life of the bridge. Flexible bridges may deform significantly and introduce significant secondary stresses that lead to premature failure of connections. To maintain a bridge for indefinite period, live load demands of frequently passing vehicles need to be kept below a certain threshold. For example, live load stress range on a fatigue details due to frequently passing vehicle should not exceed the maximum allowable fatigue stress limit. This is referred to as serviceability requirements. This requirement is incorporated within design specification and MBE. Since the intent of this process is to maintain the bridge indefinite period, rating obtained by this process yields the inventory rating.

Typically, the bridge owners use the design vehicles (HS20 loading for ASR and LFR methods and HL93 loading for LRFR methods) as the rating vehicles when establishing the inventory rating of a bridge using the strength limit state and uses the HS20 truck as rating vehicle when establishing the inventory rating of a bridge using the service limit state. Currently, the Federal Highway Administration allows bridge owners to report the inventory rating of bridge based on either HS20 or HL93 loading.

Less frequent vehicles may not affect the fatigue life or serviceability of a bridge, and thus, live load capacity available for less frequent vehicles need not be estimated using serviceability criteria. As a result, serviceability limits are not typically considered when establishing operating rating. Since permit trucks travel infrequently, all state agencies have developed their permit policies based on "operating" principles.

Agencies differ in their approach in handling the legal trucks traveling on bridges. A few agencies have developed policies to handle legal trucks based on "operating" principles, and some other agencies have developed policies based on "inventory" principles. And there are some other agencies that have developed policies that are based on the level that is in between the two levels. However, current MBE does not provide this option within LRFR method.

15.2.4 Structural Limit States

In the ASR approach, when a portion of a structural member is stressed beyond the allowable stress, the structure is considered as failed. The allowable stress is always set below yield stress of the material and therefore no portion of the structural member material ever goes into plastic deformation range and the permanent deformation is always prevented. Thus, the serviceability of a bridge is assured when the allowable stress method is used to check a bridge member. In other words, in the ASR approach, serviceability and strength criteria are satisfied automatically. The inventory and operating allowable stresses for various types of failure modes are given in MBE (AASHTO 2013).

In the LFR and LRFR approaches, failure could occur at two different limit states: serviceability and strength. When the factored load on a member reaches the ultimate capacity of the member, the structure is considered failed at its ultimate strength limit state. When the structure reaches its maximum allowable serviceability limits, the structure is considered failed at its serviceability limit state. In LFR and LRFR approaches, satisfying one of the limit states will not automatically guarantee the satisfaction of the other limit state. Thus, both serviceability and strength criteria need to be checked in the LFR and LRFR methods. Since serviceability limit analysis is performed using high frequent loading and the operating principle is typically applicable for less frequent loading, the serviceability limits need not be checked when evaluating bridges at operating level.

15.2.5 Critical Locations

The highway bridges that exist in the U.S. inventory have been designed using various highway loading and specifications. For example, bridges built in 1920s were designed using H10 loading and allowable stress method, and the bridges built in 2010s were designed using HL93 loading and load and resistance factor design. Designer of record has to meet the requirements of the Specification of design year only. However, a rating engineer needs to evaluate those bridges using the current specification. Many of the older bridges that met the requirements of the specification of the design year may not meet many of the articles of the current specifications, not only at the "maximum" demand region but also in other regions. As a result, rating engineers need to check the girders at several locations along the girder. For example, the moment envelope diagram of H10 loading and HL93 loading differs significantly (see Figure 15.2). As a result, rebar layout that satisfy the H10 loading may not be adequate for HL93 not only at maximum moment region but also at low demand regions.

Identifying the critical locations that do not meet the current specification is a difficult task. Typically, drawing the capacity of the member on top of maximum demand envelope can be done to establish the weakest location of the member. For concrete girder bridges, the change in section varies quite often as a result of rebar termination or shear reinforcement spacing and therefore plotting capacity diagram of concrete bridges becomes a difficult task. Furthermore, in some cases, the capacity depends on the applied demand. As a result, rating of a member is very complex and identifying the weakest location of the members could only be obtained by comparing demand and capacity at all the section changing points and maximum demand locations. This leads us to analyze the member at several locations. This could be overwhelming and therefore engineers should use engineering judgment in defining the

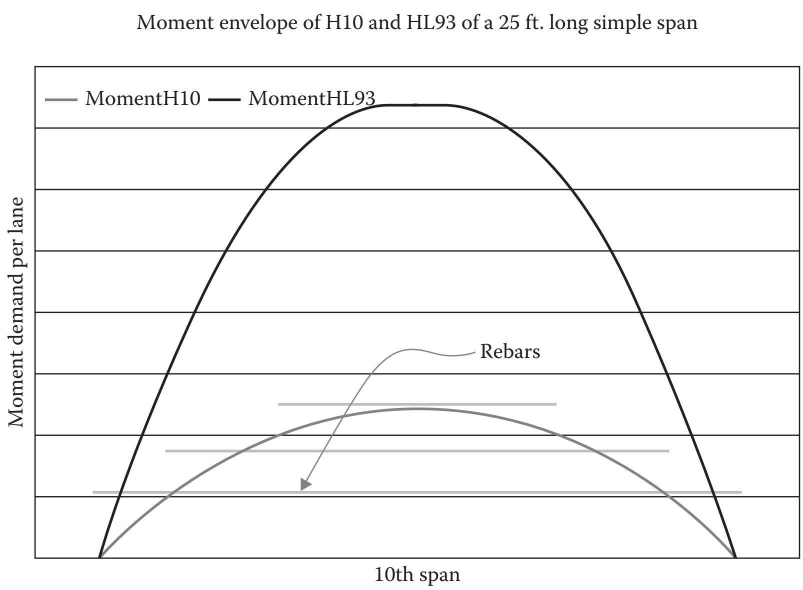


FIGURE 15.2 Moment envelopes of a girder bridge.

number of analysis points. Typically, during the design phase, the designer establishes the size of a member using the demands obtained at every 10th Point of the member span. Just like in the design of bridges, engineers typically perform the rating analysis at every 10th Point of the span.

15.2.6 Special Considerations

As stated in Section 15.2.5, older bridges may not meet the current specification requirements, and therefore, when evaluating using the current specification, these older bridges may yield lower rating values. It is important to recognize the fact that though some of the bridges yield lower rating values, they may have been performing satisfactorily without showing any distress. Engineers should use caution when establishing the rating factor for those bridges. There are many possible explanations for this behavior; two of those possible reasons are (1) analysis method is conservative and (2) material strength used for the evaluation is lower than in situ strength.

Typically, analysis method is based on the requirement listed in the design specification where the "worst" loading case scenario is considered. The LLDF to estimate the live load demand is taken conservatively; many researches performed on the live load distribution show that LLDF used in the past is conservative. Furthermore, trucks traveling within actual traffic lanes on the bridge may induce much lower demand on the girders. Also, the resistance equation listed for the capacity of a section is very simplified and conservative. For example, although, recently, researches indicate that the composite concrete decks participate in carrying shear demand, the resistance equation listed to establish shear capacity of girders ignores the contribution from concrete deck. These examples illustrate that rating engineers need to carefully consider the failure mechanism and how the resistance or live loading is determined while evaluating bridges for safe live load capacity.

Typically, evaluation of the member capacity is based on the specified minimum strength of the material. While the material properties of steel are relatively less variable, properties on concrete vary significantly. Therefore, the capacity of concrete members, where concrete strength has significant effect (e.g., shear) on the capacity, may have significant difference. Therefore, it is recommended to consider the variation of concrete strength, especially when a bridge does not show any distress but produces lower calculated rating.

15.2.6.1 Development Length Distance for Reinforcing Bars

In concrete girder bridges, a rebar does not become effective immediately at the termination point or starting point in carrying the load. A bar is considered "fully developed" at a distance of development length from its actual termination point. When designing concrete girders, designers are required to extend the rebars beyond its theoretical cutoff point by a distance of the corresponding development length. The development length depends on many parameters such as concrete strength, rebar strength, area of rebar, depth of concrete below the rebar, and epoxy coating.

When evaluating existing concrete bridges, rating engineers need to recognize the fact that concrete strength usually gets stronger as it ages and therefore the rebar development length established using the equation given in the design specification may be conservative. Also, the specification increases the basic development by a factor of 1.4 whenever the depth of concrete below the bar is more than 12 in. This is because of the possibility of fine aggregate float to the top and effectively reduces the bond strength between the rebar and concrete. In many states, concrete for reinforced concrete girder web and soffit (of box girder) is poured first, and later, once the concrete in web and soffit is hardened, concrete for deck is poured. As a result, depth of concrete pour in the second construction pour stage is usually less than the 12 in. threshold given in the Standard and LRFD Specification. Therefore, the factor of 1.4 need not be considered in establishing the development length. As a result, engineers need to use caution when establishing the development length of rebars.

Another important fact that should be considered by rating engineers is the contribution of partially developed rebars. Typically, during design phase, it is recommended to ignore the contribution of rebars that are outside of "fully developed" region. However, during rating phase, especially for bridges that show lower rating, contribution of bars that are *partially* developed could improve the overall rating of the member. Therefore, it is recommended to consider the effective portion of the rebar for evaluation. The effectiveness of a bar with a development length of L_d at a distance $X(<L_d)$ from its termination point could be taken as X/L_d.

15.2.6.2 Shear Reinforcement Spacing for Evaluation

The shear capacity due to vertical shear reinforcement depends on the stirrups' spacing. Both LFR and LRFR estimate the average area of shear stirrups that crosses the shear failure plane to establish the capacity. S is taken as the spacing of stirrups at the analysis point.

Shear capacity due to shear reinforcement is given as follows:

$$\text{For LFR method,} \quad V_s = \frac{A_v d_v}{S} f_y \tag{15.10}$$

$$\text{For LRFR method,} \quad V_s = \frac{A_v d_v \times \cot(\theta)}{S} f_y \tag{15.11}$$

where A_v = area of shear reinforcement within stirrup spacing S, f_y = specified minimum yield strength of shear reinforcement, d_v = effective shear depth, θ= angle of shear failure plane.

However, if the stirrup spacing varies within the failure plane, it is not clear as to the value to be used to establish the capacity. Figure 15.3 shows an analysis point A that is very close to the point where the shear stirrup spacing changes. Most engineers would utilize the largest spacing found in the region and conservatively obtain the capacity.

However, whenever software is utilized, this conservatism could not be expected at all times. Most software use whatever the stirrup spacing given at the point of analysis. Software would have used the S_1 for analysis point A and S_2 for analysis point B. Since S_1 is greater than S_2, the shear capacity estimated at location A will be lesser than that is estimated at point B. However, if the shear capacity is evaluated using the number of bars that cross the failure plane, there may not be significant difference in the capacity established at analysis points A and B.

Here, an equation for average stirrup spacing S, assuming the shear failure plane crosses at mid-depth of the section with the slope θ, which could be used in Equations 15.10 and 15.11, is developed.

The number of shear bar legs that are spaced at S_1 and S_2 crosses the shear failure plane are given by a/S_1 and b/S_2, respectively.

$$\frac{d_v \cot\theta}{S} = \frac{a}{S_1} + \frac{b}{S_2} = \frac{S_1 b + S_2 a}{S_1 S_2}$$

$$S = \frac{S_1 S_2}{S_1 b + S_2 a} d_v \cot\theta \tag{15.12}$$

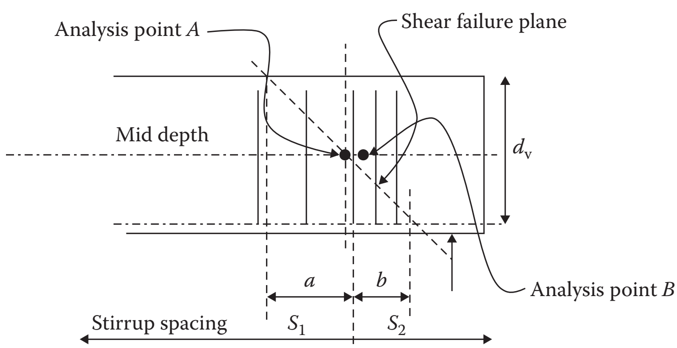


FIGURE 15.3 Shear failure plane.

When the angle of shear plane is 45° (in the case for LFR method), the preceding equation becomes $S = \dfrac{S_1 S_2}{S_1 b + S_2 a} d_v$.

When Equation 15.12 is substituted in Equations 15.10 and 15.11 for LFR and LRFR methods, the shear capacity due to stirrups becomes

$$V_s = \frac{A_v (S_1 b + S_2 a)}{S_1 S_2} f_y \tag{15.13}$$

15.2.6.3 Effect of Live Load Negative Moment Demand in Primarily Positive Dead Load Moment Region

In most continuous bridges, live loads moving along the bridge will create positive and negative effects at every point of the bridge. During the rating of a bridge, engineers try to establish the load limit that will "fail" the bridge using a vehicle configuration. For example, Figure 15.4 shows the positive and negative moment effects in the girder when the live load vehicle is in span 2. When a moving live loading vehicle is in span 2, negative demand is generated in entire girder portion of spans 1 and 3. In many older bridges, there is not enough rebars placed at top of the girders, and therefore, when rating analysis is performed using negative moment demand within spans 1 and 2, it may yield lower rating factor. In some cases, it may be controlling the overall rating of the bridge. However, rating engineers should consider the possible failure mechanism before establishing the controlling rating of the bridge. Failure mechanism due to the negative moment induced by the live load in span 2 requires the cracks in deck open up and girder deflects upward. Unless the live load vehicle stays in span 2 for significant length of time without moving, it is very unlikely that the girders will deflect upwards until failure. And, as soon as the trucks move away from span 2, the demand will get reversed and any cracks formed on top side of the deck will close as deck returns to its original stage. Since typical traffic on the bridge is moving loads, this type of failure is "unlikely" and therefore engineers should utilize the judgment when evaluating the bridges. Therefore, when the controlling rating is based on negative moment within predominantly positive moment region, engineers should use caution before finalizing the controlling rating factor. Engineers should consider the accuracy of the estimated live load demand, duration of the load demand, and negative moment capacity estimation used to establish the rating factor.

The simplified LLDF equation listed in the specification is developed by considering the largest demand at a point. Typically, to obtain maximum demand at locations, live loading is placed at around the "concerned" point. For example, trucks are placed in span 2 to obtain the LLDF for positive moment in span 2. The negative demand generated by these vehicles in the predominantly positive moment regions of spans 1 and 3 was not considered when developing the LLDF. Finite element studies show that the negative demands generated by the trucks in adjacent spans are much lower than that is established by the simplified method given in the specification. Because of conservative approach of establishing the demand, lower rating based on negative moment demand in predominantly positive moment may be too conservative.

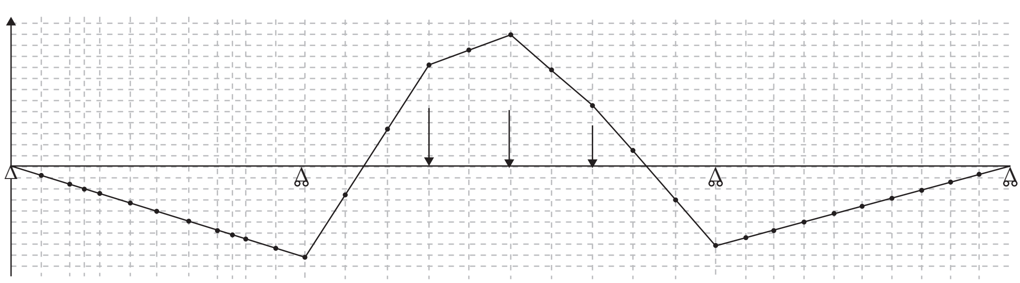


FIGURE 15.4 Moment and displacement diagram when the vehicle is in span 2 of three-span continuous bridge.

Furthermore, as soon as the live loading moves away from the adjacent spans, the negative moment will quickly dissipate by the positive moment from permanent dead loads. Therefore, the duration and nature of the live loading should be considered when establishing the bridge using the rating negative moment within the predominantly positive dead load moment regions.

Typically, engineers conservatively ignore all longitudinal deck steel (#4 or #5 bars) shown in the plans. This is because of the fact that these #4 and or #5 bars are considered to be "temperature steel" and their fundamental job is to resist the expansion and contraction forces caused by temperature variations. However, whenever "excluding" all of these bars from analysis yields lower rating of the bridge, engineers could probably consider some of the bars to improve the overall rating of the bridge.

15.2.6.4 Effect of Live Load Positive Moment Demand in Primarily Negative Dead Load Moment Region

Just like the live load negative moment in primarily positive moment region should be treated with caution, engineers should treat the live load positive moment demand in the primarily negative moment region. The following facts should be considered when establishing the rating factor in this region. The simplified LLDF that is being used to establish positive moment in negative moment region is conservative. As soon as the live loading moves away from its critical location, the dead load will reverse the effect and close any potential cracks developed by the positive moment live loading.

15.2.6.5 Evaluation Point for Negative Moment of Continuous Span Bridge

In most continuous concrete girder bridges, a question arises as to whether the rating should be performed at the center of middle support (bent or pier) or at the face of the support. Typically, most continuous concrete girder bridges have reinforced concrete diaphragm at support locations. Consider sections A and B in Figure 15.5.

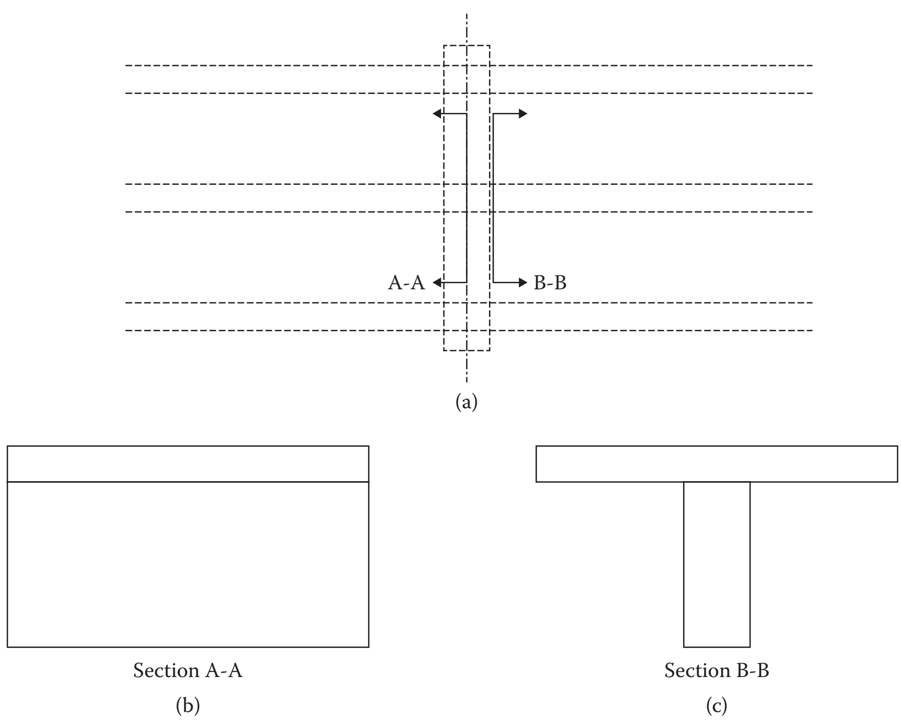


FIGURE 15.5 Details at integral bent location: (a) Plan view at bent or pier, (b) Section taken at the centerline of bent, (c) Section taken at the face of the support.

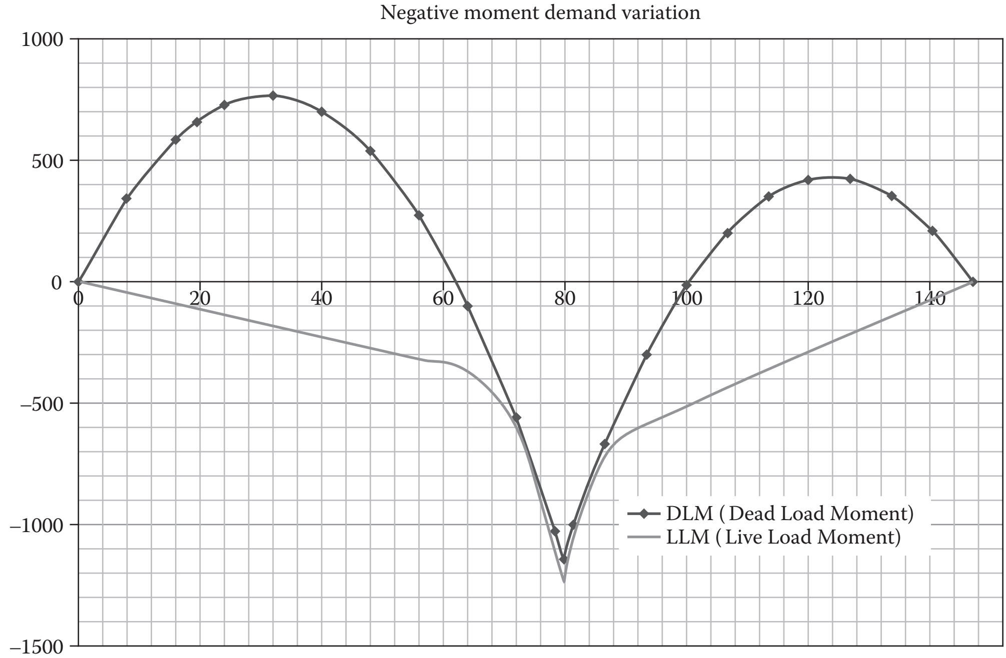


FIGURE 15.6 Typical negative moment variation over the interior support or bent.

As it can be seen from Figure 15.5, section A-A at the mid-support is a rectangular section and section B-B at the face of the diaphragm is a Tee section, where *b* is the bottom flange width. As a result, negative moment capacity based on section A-A is always greater than that is based on section B-B.

Typically, during the design phase, engineers use the negative demand reported at the centerline of support (or section A-A) but estimate the required rebars using section B-B. This typical procedure results in a conservative design. However, when evaluating existing bridges, use of this approach is not advisable since it could yield lower rating factor, especially if the bridge is designed using lighter vehicles in the past. The negative moment demand drops significantly within 3–4 ft. away from support (see Figure 15.6). In the example shown in Figure 15.6, the negative moment demand within 1.5 ft. away from the support is dropped by 10%.

Most software rate the continuous span bridges over the support using the typical girder section and therefore will yield lower rating. It is prudent for engineers to either direct the software to rate the bridge at the face of the support or utilize the correct section property at the support location so that reasonably correct rating could be achieved.

15.2.6.6 Effect of Deck Condition on Evaluation of Girders

The deck materials typically used in the bridge structures are concrete, timber, and steel. The primary function is to distribute the vehicular loads to girders. However, concrete deck that is composite with girders is not only distributes the loads but also participates as part of load carrying member. Therefore, it is important to consider the defects noted on the concrete deck in estimating the capacity of the member.

15.2.6.7 Horizontal Shear between Deck and Concrete Girder

In many of composite concrete girder structures, design specification requires that full transfer of horizontal shear forces shall be assured at contact surfaces of interconnected elements. However, this horizontal shear requirement is not routinely used to establish the rating of a bridge. One of the main reasons is that the rating engineers do not know the condition of the interface between the girder and deck. The contact surface during the construction has an important role in its capacity. For example, if the interface is clean, free of laitance, and intentionally roughened to a full amplitude of approximately 1/4 in. with minimum ties, horizontal interface shear capacity can be as high as $350b_vd$ (AASTHO, 2002). On the contrary, if the interface is not roughened, the horizontal interface shear capacity is limited to $80b_vd$ (AASTHO, 2002); there is about 75% drop in capacity. Furthermore, if the surface is not clean, the shear capacity could be much lesser than $80b_vd$ (AASTHO, 2002).

Older bridges that were designed using lighter vehicles such as H10 or H20 loading may not have adequate strength at the interface to carry larger permit trucks. Therefore, it is prudent to check the adequacy of the interface before allowing larger permit trucks. Since most bridges are "designed" with the assumption that the interface will be clean, free of laitance and intentionally roughened interface will be achieved during construction phase, the interface capacity check could be performed assuming the best case scenario. In case the rating analysis check with the best case scenario shows inadequate connection for larger permit truck, engineers should use caution in allowing those larger permit trucks.

15.2.6.8 Bridges without Plans

There are some old bridges in service without plans. Establishing safe live load capacity is essential to have a complete bridge document. When an inspector comes across a bridge without plans, sufficient field physical dimensions of each member and overall bridge geometry should be taken and recorded. In addition, information such as design year, design vehicle, designer, live load history, and field condition of the bridge need to be collected and recorded. This information will be very helpful to determine the safe live load capacity. Also, bridge inspectors need to establish the material strength using either the design year or coupon testing.

Design vehicle information could be established based on the designer (state or local agency) and the design year. For example, all state bridges have been designed using HS20 vehicle since 1944 and all local agency bridges have been designed using either H15 loading or larger than H15 loadings since 1950.

In concrete girder bridges, field dimensions help to estimate the dead loads on the girders. Since the area of reinforcing steels is not known or difficult to establish, determining the safe live load poses challenges to the bridge owners. The live load history and field condition of a bridge could be used to establish the safe load capacity of the bridge. For example, if a particular bridge has been carrying several heavy vehicles for years without damaging the bridge, this bridge could be left open for all legal vehicles.

15.3 Superstructure Rating Examples

15.3.1 General Descriptions

In this section, several problems are illustrated to show the concrete bridge rating procedures. In the following examples, the AASHTO Standard Specification for Highway Bridges, 17th Edition (AASHTO 2002), is referred as *Standard Specifications*; the AASHTO LRFD Bridge Design Specifications, 6th Edition (AASHTO (2012), is referred as 2012 *LRFD Specifications*; the AASHTO LRFD Bridge Design Specifications, 4th Edition (AASHTO 2007), is referred to as 2007 *LRFD Specifications*; and the AASHTO The Manual for Bridge Evaluation, 2nd Edition (AASHTO 2013), is referred to as *MBE*. All the notations used in these examples are defined in *Standard Specifications*, *LRFD Specification*s, or *MBE*.

In the following examples, the bridges are rated using the guidelines given in MBE using LFR and LRFR methods for the standard design vehicles and two permit trucks shown in Figure 15.7. The standard design vehicles for LFR and LRFR methods are HS20 loading and HL93 loading, respectively. It is assumed that the 5-axle permit trucks (P5 Truck—Figure 15.7a) operate with annual permit and the 13-axle permit truck (P13 Truck—Figure 15.7b) operates with single permit issued by bridge owner, but travel with other traffic. The average daily truck traffic (ADTT) of all bridges is considered to be greater than 5000.

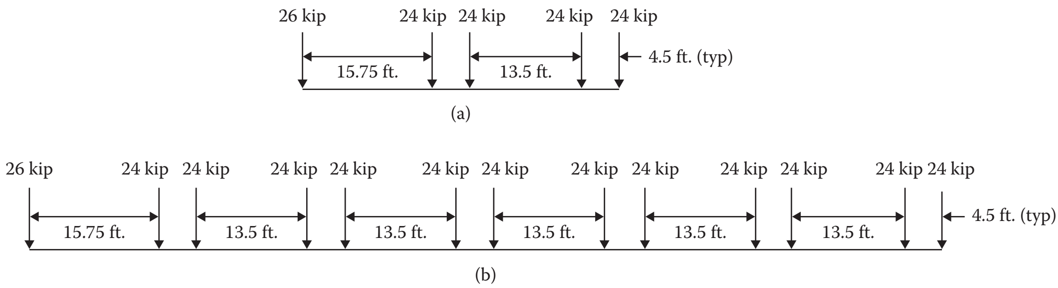


FIGURE 15.7 Permit trucks used in the rating examples: (a) 5-Axle routine permit truck configuration (P5 truck), (b) 13 Axle single permit truck configuration (P13 truck).

The general equations described in Equations 15.7 through 15.9 are modified and listed in MBE (Equations 6B.4.1.-1 and 6A.4.2.1.-1) as shown in Equations 15.14 and 15.15 for LFR and LRFR, respectively.

$$\text{LFR rating factor} = \frac{C - A_1 D}{A_2\, L(1+I)} \tag{15.14}$$

where A_1 is 1.3 for dead loads and A_2 is 1.3 and 2.17 for design loading of HS20 at operating and inventory levels, respectively. A_2 is 1.3 for permit loading.

$$\text{LRFR rating factor} = \frac{C - \gamma_{DC} DC - \gamma_{DW} DW \pm \gamma_P P}{\gamma_L\, L(1+I)} \tag{15.15}$$

where $C = \varphi\varphi_c\varphi_s R_n$, DC = dead load effect due to structural components & attachments, DW = dead load effect due to wearing surface & utilities, L = live load effect, I = dynamic load allowance, P = permanent loads other than dead loads, but $\varphi_c\varphi_s \geq 0.85$.

While the load factor for the permit trucks within LFR method is constant, the load factor for permit trucks within LRFR method depends on the ADTT and condition with which the permit truck travels on the bridge. The load factors for the permit truck that should be used for LRFR method are listed in the Table 6A.4.5.4.2a-1 of MBE.

For routine or annual permits, the load factor depends not only on ADTT but also on its gross weight. The gross weight of P5 annual permit truck is 122 kip, which is greater than 100 kip but less than 150 kip. Therefore, the load factor of P5 is obtained by interpolating the values listed for 100 and 150 kip. The load factor for a permit truck with Gross Vehicle Weight (GVW) of 100 and 150 kip is 1.8 and 1.3, respectively, for ADTT of 5000 or more.

Then, the load factor γ_L for P5 annual truck $= 1.8 - \left(\dfrac{1.8 - 1.3}{150 - 100}\right) \times (122 - 100) = 1.58.$

For special, single-trip P13 truck that travels with other trucks where ADTT is 5000 or more, according to the Table 6A.4.5.4.2a-1, the load factor γ_L is 1.50.

In the following examples (Sections 15.3.2 to 15.3.7), the load factor of 1.58 and 1.50 will be used for P5 annual permit truck and P13 single permit truck, respectively. (Recent studies suggest that these values could be lowered and there is a proposal to revise the values listed in MBE.)

As per *LRFD Specifications*, the condition factor is used to account for increasing uncertainties in a bridge member once its condition deteriorates. The condition factor is equal to 0.85 for members in poor condition, 0.95 for members in fair condition, and 1.0 for members in good condition. The system factor accounts for the level of redundancy in the structure. In these following examples, it is assumed that the bridges are in good condition and are redundant. As a result, $\varphi_c\varphi_s$ is taken as 1.0 for all the examples listed in this chapter.

15.3.2 Simply Supported T-Beam Concrete Bridge

Given: A bridge, which was built in 1929, consists of three simple span reinforced concrete T-beams on concrete bents and abutments. The span lengths are 16 ft. (4.88 m), 50 ft. (15.24 m), and 10 ft. (3.05 m). Typical cross section and girder details are shown in Figure 15.8. General notes given in the plan indicate that $f_c = 1{,}000$ psi (6.9 MPa) and $f_s = 18{,}000$ psi (124.1 MPa). Assume the weight of each barrier rail as 250 lb/ft. (3.6 N/mm).

Requirement: Assuming no deterioration of materials occurred, determine the critical rating factor of the interior girder of the second span (50 ft. or 15.24 m) of the bridge for the live load vehicles listed in Section 15.3.1 using (1) LFR method (2) LRFR method.

Solution

A. LFR Method

A.1. Dead Load Calculations

Self-weight of the girder = (3.5)(1.333)(0.15) = 0.700 kip/ft.

4 × 4 in. fillets between girder and slab = 2(1/2)(4/12)(4/12)(0.15) = 0.017 kip/ft.

Slab weight (based on tributary area) = (6.667)(8/12)(0.15) = 0.667 kip/ft.

Contribution from barrier rail (equally distributed among girders) = 2 (0.25/3) = 0.167 kip/ft.

Thus, total uniform load on the interior girder = 1.551 kip/ft. (22.6 N/mm)

Dead load moment at midspan $wL^2/8 = 484.6$ kip-ft. (0.657 MN-m)

Dead load shear at a distance d (46.25 in.) from support $w(L/2 - d) = 32.80$ kip (145.9 kN)

A.2. Live Load Calculations

The traffic lane width of this bridge is 24.0 ft. According to MBE, any bridge with a minimum traffic lane width of 18 ft. needs to carry two lanes. Hence, the LLDF should be based on two traffic lanes.

From Table 3.23.1A of Design Specification for two traffic lanes for T-beams is given by S/6.0

Number of live load wheel line = 6.667/6.0 = 1.111

AASHTO Standard impact factor for moment = 50/(125 + 50) = 0.286

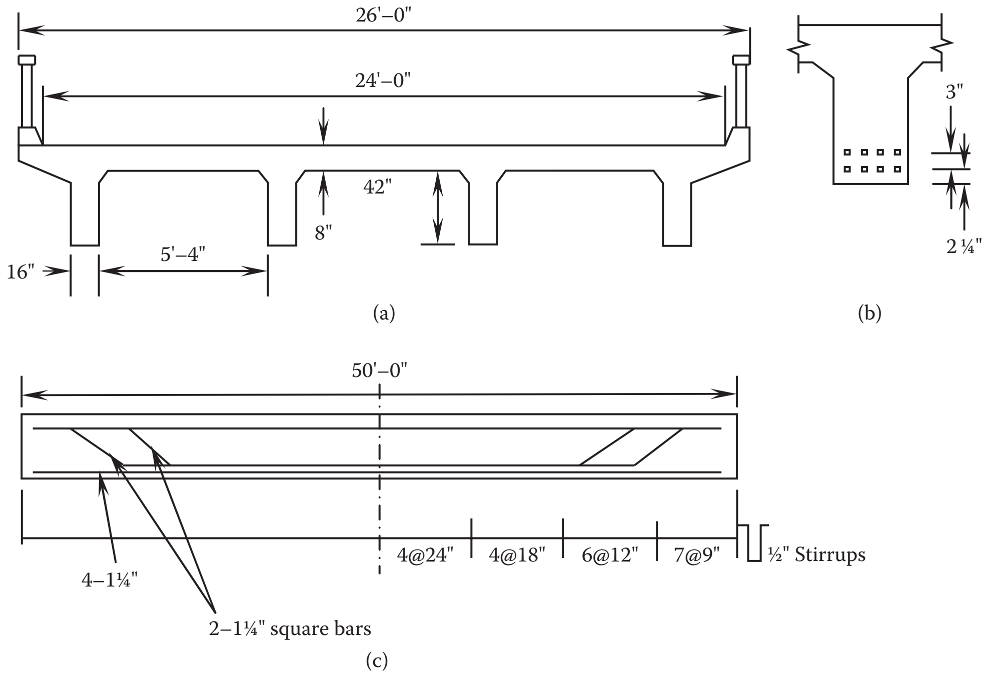


FIGURE 15.8 Details of simply supported T-beam concrete bridge example: (a) Typical cross section, (b) reinforcement locations, and (c) T-beam girder details.

TABLE 15.1 Live Load Demand at Midspan and "*d*" Distance from Support

Live Load	Midspan (Moment + *I*) per Wheel (kip-ft.)	Midspan (Moment + *I*) per Girder (kip-ft.)	End (Shear + *I*) per Wheel Line (kip)	End (Shear + *I*) per Girder (kip)
HS20	403.8	448.6	34.26	38.06
Routine permit P5 truck	533.8	593.1	45.95	51.05
Special permit P13 truck	569.3	632.6	48.19	53.54

AASHTO Standard impact factor for shear at support = 50/(125 + 50) = 0.286

The live load moments and shear tables listed in MBE are used to determine the live load demand.

Maximum HS20 moment for 50 ft. span without impact/wheel line = 314.0 kip-ft.

Thus, HS20 moment with impact at midspan = (1.286)(1.111)(314.0) = 448.6.kip-ft. (0.61 MN-m)

Maximum HS20 shear at a distance *d* from the support without impact/wheel line = 26.64 kip

Thus, maximum HS20 shear with impact = (1.286)(1.111)(26.64) = 38.06 kip (169.3 kN)

Maximum demand using one wheel line of P5 truck and P13 truck is first established using a two-dimensional analysis and then the live load demand per girder is established using LLDF and is reported in Table 15.1.

A.3. Capacity Calculations

Strengths of concrete and rebars are first determined using the guidelines provided in MBE Articles 6A.5.2.1, 6A.5.2.2, 6B.5.2.3, and 6B.5.2.4.

$f_c' = \dfrac{f_c}{0.4}$ and then $f_c' = 2{,}500$ psi and $f_s = 18{,}000$ psi and therefore $f_y = 33{,}000$ psi

A.3.1. Moment Capacity at Midspan

Total area of the steel (note that these bars are 1¼ square bars) = (8) (1.25) (1.25) = 12.5 in.^{2}

Centroid of the rebars from top deck = 42 + 8 − 3.75 = 46.25 in.

$$\text{Effective width of the deck } b_{\text{eff}} = \text{minimum of } \begin{cases} 12ts + bw = 112\text{ in.} \\ \text{Span}/4 = 150\text{ in.} \\ \text{Spacing} = 80\text{ in. } (\textit{Controls}) \end{cases}$$

$$\text{Uniform stress block depth} = a = \frac{A_s f_y}{0.85 f_c' b_{\text{eff}}} = 2.426\text{ in.} < t_s = 8\text{ in.}$$

$$\varphi M_n = \varphi A_s f_y \left(d - \frac{a}{2}\right) = 0.9 \times 12.5 \times 33 \times \left(46.25 - \frac{2.426}{2}\right) \times \left(\frac{1}{12}\right) = 1393.3\text{ kip} - \text{ft.}\,(1.88\text{ MN-m})$$

A.3.2. Shear Capacity at Support

According to AASHTO Specifications, shear at a distance *d* (50 – 2.25 – 1.5 = 46.25 in.) from the support needs to be designed. Thus, girder is rated at a distance *d* from the support. From the girder details, it is estimated that ½-in. φ stirrups were placed at a spacing of 12 in. and two 1¼ square bars were bent up.

Shear capacity due to concrete section can be calculated using two ways. In this example, the simplified method is used to establish the contribution of the concrete.

$$V_c = 2\sqrt{f_c'}\, b_w\, d = 2\sqrt{2500} \times 16 \times 46.25 \times \left(\frac{1}{1000}\right) = 74\text{ kip } (329\text{ kN})$$

Shear capacity due to shear reinforcement

$$V_s = 2 A_v \frac{f_y d_s}{S} = 2 \times 0.20 \times \frac{33 \times 46.25}{12} = 50.88\text{ kip } (226\text{ kN})$$

Shear capacity due to bent-up bars reinforcement

$$V_{sb} = 2\,A_{bar}\,f_y \sin(45) = 2 \times (1.25)^2 \times 33 \times \sin(45) = 72.9 \text{ kip (324 kN)}$$

Total shear capacity $\varphi V_n = \varphi\,(V_s + V_{sb} + V_c)$
= 0.85 (74.0 + 72.9 + 50.88) = 168.1 kip (747 kN)

A.4. Rating Calculations

$$\text{RF} = \frac{C - A_1 D}{A_2\,L(1+I)} \tag{15.14}$$

According to MBE, A_1 is 1.3 and A_2 is 2.17 and 1.3 for inventory and operating level factors, respectively. By substituting these values and appropriate load effect values, the moment and shear ratings are estimated and the results are given in Table 15.2.

A.5. Summary

Critical rating of the interior girder will then be 0.78 at inventory level and 1.31 at operating rating level for HS20 vehicle. Critical rating for P5 truck and P13 truck is 0.99 and 0.93, respectively. These critical ratings are based on moment at midspan.

B. LRFR Method

B.1. Dead Load Calculations

Dead load demand estimation of the interior girder is based on the tributary area of the girder and has been done in Section A.1 of this example.

B.2. Live Load Calculations

The traffic lane width of this bridge is 24.0 ft. According to MBE, any bridge with a minimum traffic lane width of 18 ft. needs to carry minimum of two lanes. Furthermore, the demand of the special permit trucks (in this case 13-axle P13 truck) needs to be established using the single-lane LLDF without the multiple presence factor (MPF).

Both one design lane loaded LLDF (g_1) and two or more design lanes loaded LLDF (g_2) are first established for moment and shear.

LLDF equations for moment demand of concrete T-beams listed in Table 4.6.2.2.2.b-1 of the LRFD is as follows:

One design lane loaded:

$$g_{1M} = 0.060 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

TABLE 15.2 LFR Calculations of Simply Supported T-Beam Concrete Bridge Example

Description	Midspan Moment	Shear at Support
Inventory rating	$\frac{1393.3 - 1.3 \times 484.6}{2.17 \times 448.6} = 0.78$	$\frac{168.1 - 1.3 \times 32.80}{2.17 \times 38.06} = 1.52$
Operating rating	$\frac{1393.3 - 1.3 \times 484.6}{1.3 \times 448.6} = 1.31$	$\frac{168.1 - 1.3 \times 32.80}{1.3 \times 38.06} = 2.54$
P5 rating	$\frac{1393.3 - 1.3 \times 484.6}{1.3 \times 593.1} = 0.99$	$\frac{168.1 - 1.3 \times 32.80}{1.3 \times 51.05} = 1.89$
P13 rating	$\frac{1393.3 - 1.3 \times 484.6}{1.3 \times 632.6} = 0.93$	$\frac{168.1 - 1.3 \times 32.80}{1.3 \times 53.54} = 1.80$

Two or more design lanes loaded:

$$g_{2M} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

where $K_g = I + A \times e_g^2 = \frac{1}{12} \times 16 \times 42^3 + 16 \times 42 \times (21+4)^2 = 518{,}784 \text{ in.}^3$

(Please note that the contribution of the fillets is ignored in establishing I and K_g.)

$$\text{So, } g_{2M} = 0.075 + \left(\frac{6.667}{9.5}\right)^{0.6}\left(\frac{6.667}{50}\right)^{0.2}\left(\frac{518{,}784}{12 \times 50 \times 8^3}\right)^{0.1} = 0.644$$

$$g_{1M} = 0.060 + \left(\frac{6.667}{14}\right)^{0.4}\left(\frac{6.667}{50}\right)^{0.3}\left(\frac{518{,}784}{12 \times 50 \times 8^3}\right)^{0.1} = 0.488$$

LLDF equations for shear demand of concrete T-beams listed in Table 4.6.2.2.2.b-1 is as follows:
One design lane loaded:

$$g_{1V} = 0.36 + \frac{S}{25} = 0.36 + \frac{6.667}{25} = 0.627 \text{ lanes}$$

Two or more design lanes loaded:

$$g_{2V} = 0.20 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 = 0.20 + \frac{6.667}{12} - \left(\frac{6.667}{35}\right)^2 = 0.719 \text{ lanes}$$

LLDF equations listed for moment demand of concrete T-beams in Table 4.6.2.2.2.b-1 of the LRFD Specification is valid for bridges with four or more beams.

So, the moment LLDF for the interior girder is (greater than g_{1M} and g_{2M}) = 0.644 and the shear LLDF for the interior girder is (greater than g_{1V} and g_{2V}) = 0.719.

Furthermore, MBE (Table 4.6.2.2.2.b-1) recommends that MPF of 1.2 embedded in the LLDF be removed when estimating the live load demand for "Special Permit" crossing.

So, the moment LLDF for P13 special permit truck = $g_{1M}/1.2 = 0.488/1.2 = 0.4066$.

And, the shear LLDF for P13 special permit truck = $g_{1V}/1.2 = 0.627/1.2 = 0.5225$.

The live load demands (moment and shear) for HL93, P5 truck, and P13 truck per lane (or per truck) are first established using two-dimensional analysis software. The dynamic allowance factor of 1.33 is used for all trucks. The live load demand on the girder is then established using the following equation and listed in Table 15.3.

TABLE 15.3 Critical Live Load Demands on the T-Beam Concrete Bridge Example

Live Load	Midspan (Moment + *I*) per Lane (kip-ft.)	Midspan (Moment + *I*) per Girder (kip-ft.)	(Shear + *I*) per Lane "*d*" from Support (kip)	(Shear + *I*) per Girder "*d*" from Support (kip)
HL93	1024.6	659.8	84.13	60.49
Routine permit P5 truck	1099.3	707.9	94.58	68.00
Special permit P13 truck	1173.0	477.0	99.19	51.82

Live load demand per girder = LLDF(lanes) × (live load + impact demand)/lane

B.3. Capacity Calculations

Strengths of concrete and rebars that were established in Section A.3 will be used in the following calculations.

B.3.1. Moment Capacity at Midspan

Total area of the steel (note that the bars are 1¼ square bars) $A_s = (8)(1.25)(1.25) = 12.5$ in.^{2}

Centroid of the rebars from top deck $d_s = 42 + 8 - 3.75 = 46.25$ in.

Effective width of the deck defined by Article 4.6.2.6 of 6th Edition of LRFD Specification, b_{eff} = girder spacing = 80 in.

Uniform stress block depth $= a = \dfrac{A_s\, f_y}{0.85\, f_c' b_{eff}} = 2.426 \text{ in.} < t_s = 8 \text{ in.}$

So, the neutral axis depth $c = \dfrac{a}{\beta} = \dfrac{2.426}{0.85} = 2.854.$

Then, the strain at the tensile bar $\varepsilon = \left(\dfrac{d_s - c}{c}\right) \times 0.003 = \left(\dfrac{46.25 - 2.854}{2.854}\right) \times 0.003.$

$\varepsilon = 0.0456 > 0.002$, and therefore $\varphi = 0.90$.

$$\varphi M_n = \varphi\, A_s f_y \left(d - \frac{a}{2}\right) = 0.9 \times 12.5 \times 33 \times \left(46.25 - \frac{2.426}{2}\right) \times \left(\frac{1}{12}\right)$$

$\varphi M_n = 1393.3$ kip-ft. (1.89 MN-m)

$$\text{So, } C = \varphi\varphi_c\varphi_s M_n = \varphi(1.0)(1.0) M_n$$

So, $C = \varphi M_n = 1393.3$ kip-ft. (1.89 MN-m)

B.3.2. Shear Capacity at Support

There are four alternative methods available within LRFD Specifications (Article 5.8.3.4.1, Article 5.8.3.4.3, Article 5.8.3.4.2 of 4th LRFD Edition, and Article 5.8.3.4.2 of 6th LRFD Edition) to establish the shear capacity. Please note that the general procedure (Article 5.8.3.4.2) listed in the 4th Edition has been simplified in the 5th Edition. In this example, general procedure listed (Article 5.8.3.4.2) in the 4th Edition of the LRFD Specification will be used to establish the shear capacity. Also, again, the shear capacity is established at a distance d (46.25 in.) from the support to rate the girder using shear demand.

At this location, ½-in. φ shear stirrups at a spacing of 12 in. and two 1¼ square bent bars exist.

Shear capacity due to reinforced concrete section is given by Equations 5.8.3.3-1 through 5.8.3.3-4 of LRFD and are listed in the following. Equation 5.8.3.3-1 is modified to include the contribution of bent-up bars.

$$V_n = V_c + V_s + V_{bent\text{-}up}$$

$$V_n = 0.25\, f_c' b_v d_v$$

where $V_s = \dfrac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s}$ and $V_c = 0.0316\,\beta\sqrt{f_c'}\, b_v d_v$

Effective shear depth d_v is maximum of $\begin{cases} d_e - \dfrac{a}{2} = 46.25 - 2.426/2 = 45.04 \text{ in.} \\ 0.9 d_e = 0.9 \times 46.25 = 41.63 \text{ in.} \\ 0.72\text{h} = 0.72 \times 50 = 36.0 \text{ in.} \end{cases}$

So, effective shear depth $d_v = 45.04$ in.

where d_e= effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement, h = overall depth of the member.

The angle of inclination of the shear crack (θ) and factor (β) that indicate the ability of diagonal crack to become tension crack need to be first established. The value of θ and β depends on the longitudinal strain (ε_x) at mid-depth and shear stress (ν_u) on the concrete. Longitudinal strain should be established using simultaneously acting moment demand and shear demand at that section. Therefore, the rating analysis should be performed considering two scenarios: (1) maximum shear demand and corresponding moment and (2) maximum moment and corresponding shear. However, in this example, the analysis is conservatively performed using the maximum enveloped moment and shear demands at the analysis point.

The maximum shear demands (at "*d*" distance from support) due to dead load, HL93, P5 truck, and P13 trucks are found to be 32.80, 60.49, 68.0, and 51.82 kip, respectively. The maximum moment demand (at "*d*" distance from support) due to dead load, HL93, P5 truck, and P13 trucks are found to be 137.8, 211.7, 234.8, and 155.5 kip-ft., respectively.

The longitudinal strain for reinforced concrete girders can be taken as the following equation (which is derived from the *LRFD Specification* Equation 5.8.3.4.2, by removing variables that are not applicable).

$$\varepsilon_x = \frac{\left|\frac{M_u}{d_v}\right| + 0.5V_u}{2E_sA_s} \quad \text{where} \left|\frac{M_u}{d_v}\right| \ge V_u$$

The shear stress is $v_u = \dfrac{V_u}{\varphi b_v d_v}$ where $\varphi = 0.90$, $b_v = 16$ in., and $d_v = 45.04$ in.

Typically, during rating stage, the factored demand depends on the rating factor of the live load vehicle and given by L_u = 1.25 DL + γ (RF) (LL + *I*). Since rating factor is not known, the solution will be performed by iteration procedure. The factored demand assuming rating factor of 1.00 is established first and the values for θ and β are established using Table 5.8.3.4.2-1 of the LRFD Specification and listed in Table 15.4.

Shear capacity due to bent-up bars reinforcement

$$V_{sbentup} = 2A_{bar}\ f_y \sin(45) = 2\times(1.25)^2\times 33\times\sin(45) = 72.9 \text{ kip (324 kN)}$$

Once the values for θ and β are established, the shear capacity due to concrete and steel stirrups can be established using the LRFD Equations 5.8.3.3-1 through 5.8.3.3-4. Using these estimated shear capacity contributions from concrete, shear stirrups, and bent bars, the total shear capacity is established using $\varphi V_n = \varphi\ (V_s + V_{sbent\text{-}up} + V_c)$ equation and listed in Table 15.5.

TABLE 15.4 Initial Values for θ and β Established Using Factored Demand with the Assumed RF

Load Combination	Factored Moment Demand (M_u) (kip-ft.)	Factored Shear Demand (V_u) (kip)	Actual M_u/d_v (kip)	M_u/d_v Used in the Calculation (kip) $M_u/d_v > V_u$	ε_x	$\frac{v_u}{f'_c}$	θ	β
HL93 + DL at inventory	1.25 × 137.8 + 1.75 × 211.7 = 542.7	1.25 × 32.77 + 1.75 × 60.49 = 146.8	144.6	146.8	0.000608	0.0905	32.07	2.45
HL93 + DL at operating	1.25 × 137.8 + 1.35 × 211.7 = 458.0	1.25 × 32.77 + 1.35 × 60.49 = 122.6	122.0	122.6	0.000507	0.0756	30.60	2.58
Routine P5 + DL at operating	1.25 × 137.8 + 1.58 × 234.8 = 543.2	1.25 × 32.77 + 1.58 × 68.00 = 148.4	144.7	148.4	0.000614	0.0915	32.16	2.44
Special P13 + DL at operating	1.25 × 137.8 + 1.50 × 155.5 = 405.5	1.25 × 32.77 + 1.50 × 51.82 = 118.7	108.0	118.7	0.000491	0.0976	30.34	2.61

TABLE 15.5 Shear Capacity Based on First Iteration That Is Based on the Factored Demands Established Using RF of 1.00

Load Combination	Assumed RF	θ	β	V_c (kip)	V_s (kip)	$V_{sbent\text{-}up}$ (kip)	V_n (kip)	Estimated RF
HL93 inventory	1.00	32.07	2.45	88.21	95.84	72.90	231.3	1.798
HL93 operating	1.00	30.60	2.58	92.90	101.54	72.90	240.6	2.448
Routine P5	1.00	32.16	2.44	87.85	95.51	72.90	230.6	1.765
Special P13	1.00	30.34	2.61	93.98	102.60	72.90	242.5	2.593

Using the established capacity, the rating factor for shear is established using Equation 15.15 and the estimated rating factors are listed in Table 15.5. It is important to recognize that the capacity is established assuming the rating factor of 1.00 for all vehicles. Since the established rating factor is not equal to the assumed rating factor (in this case 1.0), the capacity needs to be adjusted until the assumed rating factor and calculated rating factor matches.

By performing several iterations, final capacity is established and the final results (including assumed rating factor and estimated rating factor) are listed in Table 15.6.

B.4. Rating Calculations

$$\mathrm{RF} = \frac{C - \gamma_{\mathrm{DC}} DC - \gamma_{\mathrm{DW}} DW \pm \gamma_{\mathrm{P}} P}{\gamma_{\mathrm{L}} L(1+I)} \tag{15.15}$$

According to *MBE*, γ_{DC} is 1.25, γ_{DW} is 1.5, and, γ_{L} is 1.75 and 1.35 for inventory and operating levels, respectively. The value for γ_{L} is established before the example problems for routine P5 truck and special P13 truck. They are 1.58 and 1.50 for routine permit and special permit truck, respectively. Note that there is no loading due to wearing surface or prestressing force. By substituting demands and appropriate load factors, the moment and shear rating are established and the results are given in Table 15.7.

B.5. Summary

Critical rating of the interior girder will then be 0.68 at inventory level and 0.88 at operating rating level for HL93 vehicle. The permit rating for routine P5 truck and Special trip P13 truck are 0.71 and 1.10, respectively.

TABLE 15.6 Final Shear Capacity Based on Several Iterations Performed

Load Combination	Assumed RF	θ	β	V_c (kip)	V_s (kip)	$V_{sbent\text{-}up}$ (kip)	V_n (kip)	Estimated RF
HL93 inventory	1.609	35.77	2.18	78.49	83.36	72.90	211.3	1.609
HL93 operating	2.086	35.77	2.18	78.49	83.36	72.90	211.3	2.086
Routine P5	1.585	35.77	2.18	78.49	83.36	72.90	211.3	1.585
Special P13	2.191	35.77	2.18	78.49	83.36	72.90	211.3	2.191

TABLE 15.7 LRFR Calculations of Simply Supported T-Beam Concrete Bridge Example

Location	Moment at Midspan	Shear at End
Inventory rating	$\frac{1393.3 - 1.25 \times 484.6}{1.75 \times 659.8} = 0.68$	$\frac{211.3 - 1.25 \times 32.80}{1.75 \times 60.49} = 1.61$
Operating rating	$\frac{1393.3 - 1.25 \times 484.6}{1.35 \times 659.8} = 0.88$	$\frac{211.3 - 1.25 \times 32.80}{1.35 \times 60.49} = 2.09$
Routine permit P5 rating	$\frac{1393.3 - 1.25 \times 484.6}{1.58 \times 707.9} = 0.71$	$\frac{211.3 - 1.25 \times 32.80}{1.58 \times 68.00} = 1.59$
Special permit P13 rating	$\frac{1393.3 - 1.25 \times 484.6}{1.50 \times 477.0} = 1.10$	$\frac{211.3 - 1.25 \times 32.80}{1.50 \times 51.82} = 2.19$

15.3.3 Simply Supported Reinforced Concrete Flat Slab Bridge

Given: A single span, simply supported longitudinally reinforced, 17.0-in.-thick (430 mm) concrete slab superstructure with diaphragm abutment is supported by five 16.0-in.-diameter (406 mm) reinforced concrete piles. The span length is 23.5 ft. (7.2 m), overall width of the bridge is 43.5 ft. (13.26 m), and this bridge has been carrying two lanes of traffic. Typical cross section, elevation, and top and bottom longitudinal rebar layout are shown in Figure 15.9. General notes given in the as-built plan indicating that $f_c' = 3{,}250$ psi (22.4 MPa) and $f_y = 60{,}000$ psi (413.7 MPa). Assume the weight of each barrier rail as 400 lb/ft. (5.8 N/mm). Clear cover distances for bottom and top rebars are 1.5 and 2.5 in., respectively.

Requirement: Assuming no deterioration of materials occurred, determine the critical rating factor of the bridge for the live load vehicles listed in Section 15.3.1 using (1) LFR method and (2) LRFR method.

Solution

Typically, reinforced concrete slab system is designed using "Equivalent Slab width" (or sometimes referred to as "E" width, or "Strip width") concept, where effect of one wheel line of live loads is carried by the equivalent slab width (E). This same concept is used to rate the reinforced concrete slab bridge. Furthermore, *Standard Specifications* and *LRFD Specifications* state that a reinforced concrete slab that is designed for moment demand is considered adequate for shear. As a result, rating using shear demand will not be performed.

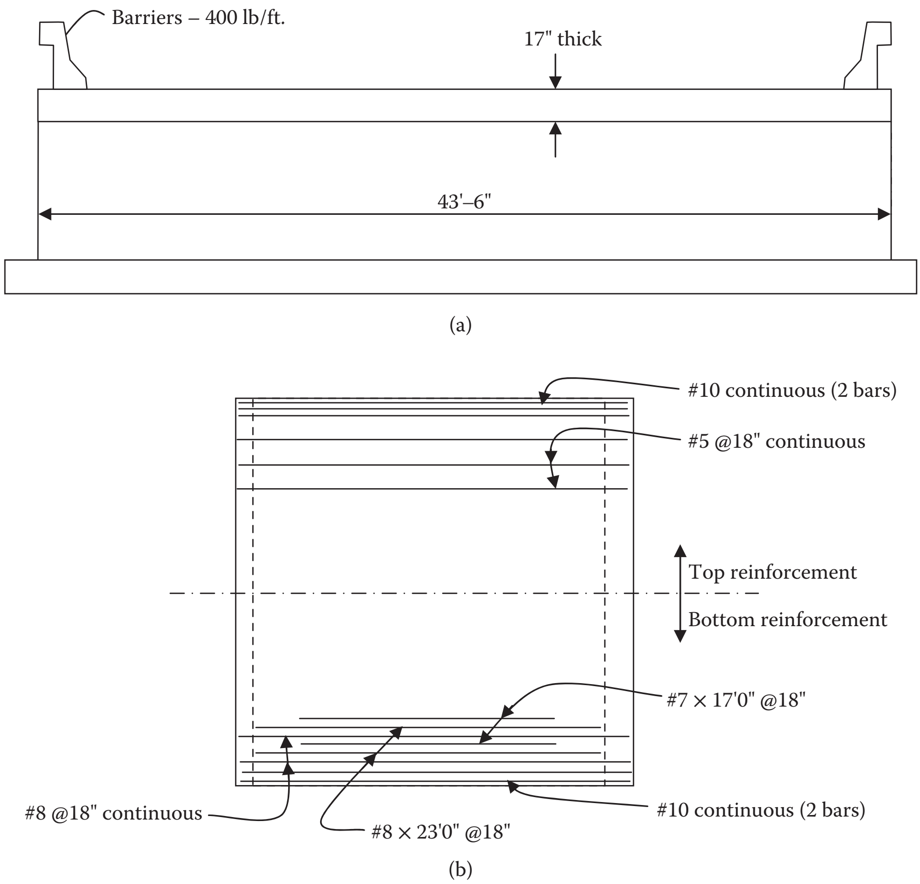


FIGURE 15.9 Details of single-span RC slab bridge example: (a) Typical section and (b) reinforcement layout of 43.5-ft.-wide single-span (23.5 ft.) RC slab.

In this example, 12 ft.-wide reinforced concrete slab is used to rate the bridge using both LFR and LRFR methods. To illustrate the rating analysis, detailed calculations are performed at midspan location (5/10th point) and 2/10th point of the span. All the necessary steps are followed at these locations. Later, the rating is performed at every 10th point of the span, but only the results are listed.

A. LFR Method

A.1. Dead Load Calculation

Self-weight of 12-ft.-wide slab = (12.0)(17.0/12)(0.15) = 2.550 kip/ft.

Barrier weight is conservatively distributed to entire slab equally and therefore the equivalent barrier weight per 12-ft. strip slab = (2)(0.400)(12)/43.5 = 0.221 kip/ft.

So, the total uniform dead load = 2.550 + 0.221 = 2.771 kip/ft.

A.2. Live Load Calculation

From Article 3.23.3 of Standard Specification, the effective slab width that carries one wheel line is 4 + 0.06S, but limited to 7 ft.

$E = 4 + (0.06)(23.5) = 5.41$ ft. (< 7 ft.)

The number of live load wheels carried by 12-ft. slab = 12/5.41 = 2.218 wheel lines

So, LLDF = 2.218 wheel lines

A.3. Live Load and Dead Load Analysis

Although analysis of simply supported superstructure can be performed by hand calculation, the dead and live load demands are obtained using two-dimensional analysis software. Since the bridge is symmetrical about midpoint of the span, results at every 10th point up to midpoint of the span are listed in Table 15.8.

A.4. Rating Calculation

Rating of bridge typically is done using the following equation:

$$\mathrm{RF} = \frac{C - A_1 D}{A_2 \times (LL + I)} \tag{15.14}$$

The values for dead and live loads have already been established. Next step in establishing the rating factor is the capacity of the member. Capacity and load demand vary along the girder independently. Therefore, to establish the lowest rating factor for the entire girder, one needs to perform the analysis in several locations. Here, in this example, rating is performed *only* at every 10th point of the span. To illustrate the details of the rating analysis, all the steps to obtain the rating at midspan location and 2/10th of the span are documented. The analysis results of other 10th points are listed at the end of this example for reference.

(a) Rating at Midspan Location

Slab bridges need to be rated for moment demand only. Furthermore, simple span structures do not experience any negative moment demand. As a result, only the positive moment capacity needs to be established to rate the bridge.

TABLE 15.8 Dead and Live Load Moment Demands on 12-ft.-Wide Slab Strip

Location	Distance (ft.)	DL_M (kip-ft.)	Positive Moment with Impact (kip-ft.)		
			HS20	P5	P13
Sp 1 – 0.0 pt.	0.00	0.0	0.0	0.0	0.0
Sp 1 – 0.1 pt.	2.35	68.9	131.2	141.5	142.3
Sp 1 – 0.2 pt.	4.70	122.5	219.0	234.2	235.8
Sp 1 – 0.3 pt.	7.05	160.7	263.4	295.2	295.2
Sp 1 – 0.4 pt.	9.40	183.6	264.5	328.5	328.5
Sp 1 – 0.5 pt.	11.75	191.3	271.0	329.3	329.3

During design phase, rebars are typically extended beyond its theoretical termination point by its required development length. Similarly, during the rating phase, the bars should be considered "fully" effective at a distance of development from the actual cutoff point. Conservatively, engineers used to ignore the structural contribution of the portion of the bars that are not "fully" developed. However, during rating procedure, engineers could take advantage of these "partially" developed bars in establishing capacity and this example consider these bars in the evaluation (see more discussion in Section 15.2.6.1).

To establish the effectiveness of rebars at a section, development length of the bars in the bridge is first established. The development of tensile rebar L_{bar} for bars lesser than #11 bars is given by Article 8.25.1 of the Standard Specification.

$$L_{bar} = \frac{0.04A_b \times f_y}{\sqrt{f_c'}}, \text{ but not less than } 0.0004d_b f_y$$

In this example, $f_y = 60{,}000$ psi, $f_c' = 3250$ psi

Article 8.25.2 of *Standard Specifications* states that whenever the depth of the concrete pours below the reinforcement is more than 12 in., the basic development length should be increased by a factor of 1.4. In this case, depth of concrete below the top rebar is 14.0 in. and therefore the factor of 1.4 needs to be included for top reinforcement of the bridge.

Article 8.25.3.1 of the *Standard Specifications* states that whenever the lateral bar spacing is 6 in. or more, this basic development length could be reduced by a factor of 0.8. In this bridge, the longitudinal slab reinforcements are spaced at 6 in. apart and therefore a factor of 0.8 could be applied for all bars. Those requirements are summarized in Table 15.9.

At midspan location, distance to the bar termination point from the midspan is much greater than the required development length for all bars and therefore all the bottom reinforcements are fully developed.

The number of #8 (23-ft.-long) bars/12-ft. slab = (12)(12)/(18) = 8 bars.

Similarly, the number of #8 (continuous) bars/12 ft. = (12)(12)/(18) = 8 bars.

Similarly, the number of #7 bars/12 ft. = (12)(12)/(18) = 8 bars.

So, the total area of rebars at midspan $A_s = 8 \times 0.79 + 8 \times 0.79 + 8 \times 0.60 = 17.44$ in.^{2}

The moment capacity of the section is given by $\varphi M_n = \varphi A_s f_y \left(d_s - \frac{A_s f_y}{0.85 f_c' b} \right)$

Centroid of the rebar location from top $d_s = D_{slab}$ – clear cover – $1/2 d_b$ = 17.00 – 1.50 – 1.00/2 = 15.00 in.

The section is a rectangular section and therefore depth of compression block $a = \frac{A_s \times f_y}{0.85 f_c' b}$

$$a = \frac{A_s \times f_y}{0.85 f_c' b} = \frac{17.44 \times 60}{0.85 \times 3.25 \times 12.0 \times 12} = 2.63 \text{ in.}$$

Then positive moment capacity

$$\varphi M_n = 0.9 \times 17.44 \times 60 \times \left(15.00 - \frac{2.63}{2} \right) \times \left(\frac{1}{12} \right) = 1074.0 \text{ kip-ft.}$$

TABLE 15.9 Required Development Length of the Existing Rebars

Bar (Location)	Area (in.^{2})	Basic Development Length (in.)	Factor for Depth	Factor for Spacing ≥6 in.	Required Development length
#5 (top bar)	0.31	13.05	1.4	0.8	14.62 in. = 1.22 ft.
#8 (bottom)	0.79	33.25	1.0	0.8	26.61 in. = 2.22 ft.
#7 (bottom)	0.60	25.25	1.0	0.8	20.21 in. = 1.68 ft.

So, $C = \varphi M_n = 1074$ kip-ft.

Rating factor based on moment at midspan location:

$$\mathrm{RF} = \frac{C - A_1 D}{A_2 \times (LL + I)} \tag{15.14}$$

where $A_1 = 1.3$ and $A_2 = 2.17$ and 1.3 for inventory rating and operating rating. $A_2 = 1.3$ for permit rating (MBE Article 6B.4.3).

The total dead load demand = 191.3 kip-ft.

$$\text{Inventory RF} = \frac{1074.0 - 1.3 \times 191.3}{2.17 \times 271.0} = 1.41$$

$$\text{Operating RF} = \frac{1074.0 - 1.3 \times 191.3}{1.30 \times 271.0} = 2.34$$

$$\text{5-axle routine permit RF} = \frac{1074.0 - 1.3 \times 191.3}{1.30 \times 329.3} = 1.93$$

$$\text{13-axle special permit RF} = \frac{1074.0 - 1.3 \times 191.3}{1.30 \times 329.3} = 1.93$$

(b) Rating at 2/10th Point of Span 1

A few bars are terminated closer to this analysis point, and therefore, as a first step, effectiveness of each bar crossing this analysis point needs to be established. Of the three bottom reinforcements, distance from this analysis point to the termination point of the #8 continuous bar and #8 bar (23.5 ft. long) are greater than the required development length and therefore #8 bars are fully effective at this point. However, it is not the case for #7 bar, and as a result, the effectiveness of the bar is established as follows:

The development length of #7 bar, $L_{dev} = 1.68$ ft. (see Table 15.9)

Length from 2/10th point to closest bar termination point $L_{d\text{-}2/10pt} = 4.7 - 3.25 = 1.45$ ft.

Therefore % of effectiveness $p \times 100 = L_{d\text{-}2/10pt}/L_{dev} = 1.45/1.68 = 0.863 \times 100\% = 86.3\%$

Table 15.10 summarizes the effectiveness of each bar and total effective rebar area at the 2/10th point of Span 1.

Centroid of the bottom rebar location from top = t_{slab} – clear cover – $1/2d_b$ = 17.00 – 1.50 – 1.00/2 = 15.00 in.

The section is a rectangular section and therefore depth of compression block $a = \dfrac{A_s \times f_y}{0.85\, f'_c b}$

$$a = \frac{A_s \times f_y}{0.85\, f'_c b} = \frac{16.78 \times 60}{0.85 \times 3.25 \times 12.0 \times 12} = 2.53 \text{ in.}$$

Then positive moment capacity

$$\varphi M_n = 0.9 \times 16.78 \times 60 \times \left(15.00 - \frac{2.53}{2}\right) \times \left(\frac{1}{12}\right) = 1037.1 \text{ kip-ft.}$$

So, $C = \varphi M_n = 1037.1$ kip-ft.

TABLE 15.10 Total Area Estimation at 2/10th Point of the Span 1

Bar Size	L_{dev} (ft.)	$L_{d\text{-}2/10th\ pt}$ (ft.)	$\%_{Effective}$ ($p \times 100$)	No. of Bars per 12-ft. Slab (n)	A_{sb} per Bar	$A_{s\text{-}bar}$ (n p A_{sb})	$A_{s\text{-}total}$ $\sum A_{s\text{-}bar}$
#8 (bottom cont)	2.22	4.70	100	8	0.79	6.32	16.78
#8 (bottom 23 ft.)	2.22	4.45	100	8	0.79	6.32	
#7 (bottom 17 ft.)	1.68	1.45	86.3	8	0.60	4.14	

Rating factor based on moment at midspan location:

$$\mathrm{RF} = \frac{C - A_1 D}{A_2 \times (\mathrm{LL} + I)} \tag{15.14}$$

where A_1=1.3 and A_2 = 2.17 and 1.3 for inventory rating and operating rating. A_2 = 1.3 for permitrating (MBE 6B.4.3).

The total dead load demand = 122.5 kip-ft.

$$\text{Inventory RF} = \frac{1037.1 - 1.3 \times 122.5}{2.17 \times 219.0} = 1.85$$

$$\text{Operating RF} = \frac{1037.1 - 1.3 \times 122.5}{1.3 \times 219.0} = 3.08$$

$$\text{5-axle routine permit RF} = \frac{1037.1 - 1.3 \times 122.5}{1.3 \times 234.2} = 2.88$$

$$\text{13-axle special permit RF} = \frac{1037.1 - 1.3 \times 122.5}{1.3 \times 235.8} = 2.86$$

(c) Rating at All 10th Points

As stated in the first paragraph of section A.4, to obtain the critical rating of the bridge, rating needs to be performed at all 10th points of the span and at points where section (in this case where ever rebar pattern changes) changes. Since this bridge is symmetrical bridge, the bridge rating is performed at all 10th points of first half of the bridge. Section change points are not considered in this illustrative example, but can be easily done by any automated software. Table 15.11 shows the rating based on positive moment. The most critical rating occurred at midspan location.

The controlling (lowest) inventory rating factor and operating rating factor for HS20 loading are 1.41 and 2.34, respectively. The operating rating factors for 5-axle permit and 13-axle permit trucks are 1.93 and 1.93, respectively.

B. LRFR Method

To make the analysis consistent between the LFR and LRFR methods, demand and capacity of 12-ft. strip are used to establish the rating factor.

B.1. Dead Load Analysis

Dead load of 12-ft.-wide slab has already been established within LFR method.

B.2. Live Load Analysis

The approximate method of analysis listed in the Article 4.6.2.3 of LRFD Specification is used to establish "Equivalent Slab width" (or "E" width) that carries one lane of live loads.

TABLE 15.11 LFR Rating Based on Positive Moment Demand at Every 10th Point

	Area of bottom reinforcement	Positive Moment Capacity	Dead Load Moment	Demand with Impact			Rating Factor			
Location	$A_{s\text{-bott}}$(in.^{2})	Pos M_{cap}(k-ft.)	DL_M (k-ft.)	HS20 (k-ft.)	P5 (k-ft.)	P13 (k-ft.)	Inventory RF	Operating RF	P5 RF	P13 RF
Sp 1 – 0.0 pt.	6.32	413.0	0.0	0.0	0.0	0.0	>99	>99	>99	>99
Sp 1 – 0.1 pt.	12.30	778.8	68.9	131.2	141.5	142.3	2.42	4.04	3.75	3.73
Sp 1 – 0.2 pt.	16.78	1037.1	122.5	219.0	234.2	235.8	1.85	3.08	2.88	2.86
Sp 1 – 0.3 pt.	17.44	1074.0	160.7	263.4	295.2	295.2	1.52	2.53	2.25	2.25
Sp 1 – 0.4 pt.	17.44	1074.0	183.6	264.5	328.5	328.5	1.46	2.43	1.96	1.96
Sp 1 – 0.5 pt.	17.44	1074.0	191.3	271.0	329.3	329.3	1.41	2.34	1.93	1.93

The width of equivalent slab width for single-lane loaded and two-or-more loaded cases is as follows:

$E_{one} = 10 + 5\sqrt{L_1 W_1}$ in. $\leq \dfrac{12W}{N_L}$ and $L_1 \leq 60$ ft. and $W_1 \leq 30$ ft. (LRFD Article 4.6.2.3-1)

and $E_{multi} = 84 + 1.44\sqrt{L_1 W_1}$ in. $\leq \dfrac{12W}{N_L}$ and L_1 and $W_1 \leq 60$ ft. (LRFD Article 4.6.2.3-2)

For this slab bridge, $N_L = 3$, $L_1 = 23.5$ ft., $W_1 = 43.5$ ft.

$$E_{one} = 10 + 5\sqrt{23.5 \times 30.0} = 142.8 \text{ in.} \leq \frac{12 \times 43.5}{3} = 174.0 \text{ in.}$$

$$E_{multi} = 84 + 1.44\sqrt{23.5 \times 43.5} = 130 \text{ in.} \leq \frac{12 \times 43.5}{3} = 174.0 \text{ in.}$$

Therefore, $E_{one} = 142.8$ in. (11.9 ft.) and $E_{multi} = 130$ in. (10.833 ft.).

The number of HL93 lanes carried by 12-ft. slab = 12/10.833 = 1.108 lanes.

The number of routine P5 truck lanes carried by 12-ft. slab = 12/10.833 = 1.108 lanes.

MBE requires that when evaluating bridges for "special permits," single-lane distribution without the MPFs should be used.

So, the number of special P13 truck lanes carried by 12-ft. slab = 12/(1.2 × 11.9) = 0.841 lanes.

The live load demands are obtained using two-dimensional analysis software. Since the bridge is symmetrical about midspan of the span, results are listed at every 10th point up to midpoint of the span in Table 15.12.

B.3. Capacity Calculation

As stated in the first paragraph of section A.4, here in this example, rating is performed at every 10th point of the span and therefore the moment capacities at 10th points are established here.

(a) Effect of Development Length

As stated in the LFR method, effect of development length of each rebar needs to be considered when establishing the effective bar area at an analysis point.

The LRFD Articles 5.11.2.1.1, 5.11.2.1.2, and, 5.11.2.3 are applicable in establishing the development length. Using these articles, the effective development and effective area at every 10th point are established and listed in Tables 15.13 and 15.14, respectively.

TABLE 15.12 Dead and Live Load Moment Demands on 12-ft.-Wide Slab Strip

			Positive Moment with Impact (kip-ft.)		
Location	Distance (ft.)	DL_M (kip-ft.)	HL93	P5	P13
Sp 1 – 0.0 pt.	0.00	0.0	0.0	0.0	0.0
Sp 1 – 0.1 pt.	2.35	68.9	158.5	143.9	109.7
Sp 1 – 0.2 pt.	4.70	122.5	278.2	238.5	181.2
Sp 1 – 0.3 Pt.	7.05	160.7	358.9	299.7	227.5
Sp 1 – 0.4 pt.	9.40	183.6	402.6	334.0	253.7
Sp 1 – 0.5 pt.	11.75	191.3	408.1	336.0	255.0

TABLE 15.13 Required Development Lengths of All Reinforcements at Midspan

Bar (Location)	Area (in.^{2})	Diameter d_b (in.)	Basic Development (in.)	Factor for Depth	Factor for Spacing ≥ 6 in.	Required Development L_{dev}
#5 (top bar)	0.31	0.625	12.90	1.4	0.8	14.44 in. = 1.20 ft.
#8 (bottom)	0.79	1.00	32.87	1.0	0.8	26.30 in. = 2.19 ft.
#7 (bottom)	0.60	0.875	24.96	1.0	0.8	19.97 in. = 1.66 ft.

TABLE 15.14 Effective Area of Rebar, A_{sbt} per 12-ft.-Wide Slab Strip at Each 10th Point

Location	Bar Size	L_{dev} (ft.)	$L_{d\text{-}p/10th\ pt}$ (ft.)	$\%_{Effective}$ ($p \times 100$)	No. of Bars per 12-ft. Slab (n)	A_s per Bar (A_{sb})	$A_{s\text{-}bar}$ ($n\ p\ A_{sb}$)	$A_{sbt} = \sum A_{s-bar}$
Sp 1 – 0.0 pt.	#8 (cont)	2.19	—	100	8	0.79	6.32	6.32
Sp 1 – 0.1 pt.	#8 (cont)	2.19	—	100	8	0.79	6.32	12.38
	#8 (23 ft.)	2.19	2.10	95.9	8	0.79	6.06	
Sp 1 – 0.2 pt.	#8 (cont)	2.19	—	100	8	0.79	6.32	16.83
	#8 (23 ft.)	2.19	4.45	100	8	0.79	6.32	
	#7 (17 ft.)	1.66	1.45	87.3	8	0.60	4.19	
Sp 1 – 0.3 pt. to Sp 1 – 0.5 pt.	#8 (cont)	2.19	—	100	8	0.79	6.32	17.44
	#8 (23 ft.)	2.19	>6.8	100	8	0.79	6.32	
	#7 (17 ft.)	1.66	>3.8	100	8	0.60	4.80	

Note: $L_{d\text{-}p/10th\ pt}$ = shortest distance from termination point to the p/10th point.

TABLE 15.15 Capacity of 12-ft. Slab Strip at Every 10th Point of the Span

Span, 10th Point	$A_{sbt=}$ (in.^{2})	c (in.)	ε_t	φ	φM_n(kip-ft.)
Sp 1 – 0.1 pt.	12.38	2.197	0.017	0.90	783.6
Sp 1 – 0.2 pt.	16.83	2.987	0.012	0.90	1040.1
Sp 1 – 0.3 pt.	17.44	3.095	0.012	0.90	1074.0
Sp 1 – 0.4 pt.	17.44	3.095	0.012	0.90	1074.0
Sp 1 – 0.5 pt.	17.44	3.095	0.012	0.90	1074.0

The basic development length for a bar lesser than #11 bars is $\dfrac{1.25A_b \times f_y}{\sqrt{f_c'}}$, but not less than $0.4d_b f_y$.

(b) Moment Reduction Factor

Within LRFD specification, the moment reduction factor varies with the tensile strain in the tensile rebar. The strain at the centroid of the tension bars can be established by the following equation (LRFD Commentary C5.7.2.1-1 and Equation 5.7.3.1.2-4).

$$\varepsilon_t = \frac{0.003(d_s - c)}{c}, \text{where } c = \frac{A_s f_y}{0.85\beta_1 f_c' b}$$

For strain $\varepsilon_t \geq 0.005$, the moment reduction factor φ is 0.90; for $0.005 \geq \varepsilon_t \geq 0.002$, moment reduction factor φ is $0.65 + 0.15\left(\dfrac{d_s}{c} - 1\right)$, and for strain $\varepsilon_t \leq 0.002$, moment reduction factor φ is 0.75.

Centroid of the rebar location from bottom $d_s = t_{slab}$ – clear cover – $1/2d_b$ = 17.00 – 1.5 – 1.00/2 = 15.00 in.

Moment capacity is first established at every 10th point of the spans. Calculations are performed and listed in Table 15.15.

B.4. Rating Calculation

The LRFR of bridge typically is done using the following equation:

$$\text{RF} = \frac{C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_P P}{\gamma_L L(1+I)} \tag{15.15}$$

The load factors for the HL93, routine permit P5 truck, and special permit P13 truck are developed in Section 15.3.1 and they are 1.75 for inventory rating, 1.35 for operating rating, 1.58 for routine P5 truck, and 1.50 special permit truck P13. The load factor γ_{DC} is 1.25 and γ_{DW} is 1.5.

TABLE 15.16 LRFR Factor of 12-ft. Slab Strip at Every 10th Point of the Span

Span, 10th Point	$C = \varphi M_n$(kip-ft.)	DL_M(kip-ft.)	HL93 Moment Demand (kip-ft.)	HL93 RF Inventory RF	HL93 RF Operating RF	P5 + I Moment Demand (kip-ft.)	P5 + I RF	P13 + I Moment Demand (kip-ft.)	P13 RF
Sp 1 – 0.1 pt	783.6	68.9	158.5	2.51	3.26	143.9	3.07	109.7	4.24
Sp 1 – 0.2 pt	1040.1	122.5	278.2	1.82	2.36	238.5	2.35	181.2	3.26
Sp 1 – 0.3 pt	1074.0	160.7	358.9	1.39	1.80	299.7	1.84	227.5	2.56
Sp 1 – 0.4 pt	1074.0	183.6	402.6	1.20	1.55	334.0	1.60	253.7	2.22

Rating factors at the 10th point of the bridge are established using MBE Equation 6A.5.1.-1 and are listed in Table 15.16.

The inventory rating and operating rating based on HL93 is 1.17 and 1.52, respectively. Permit rating for 5-axle routine permit truck and for 13-axle special permit truck are 1.57 and 2.18, respectively.

15.3.4 Three-Span Continuous T-Beam Concrete Bridge

Given: A bridge, which was built in 1966, consists of a three continuous span reinforced concrete T-beam superstructure that is supported with five 16"-diameter, 16-ft.-long reinforced concrete column bent and abutments. The bent cap width is 30 in. The span lengths are 32 ft. (9.75 m), 43 ft. (13.11 m), and 32 ft. (9.75 m). Typical cross section and interior girder details are shown in Figure 15.10. General notes given in the plan indicate that $f_c = 1{,}200$ psi (8.3 MPa) and $f_s = 20{,}000$ psi (137.9 MPa). Assume the weight of each barrier rail as 250 lb/ft. (3.6 N/mm).

Requirement: Assuming no deterioration of materials occurred, determine the critical rating factor of the interior girder the bridge for the live load vehicles listed in Section 15.3.1 using (1) LFR method (2) LRFR method.

Solution:

Before performing any analysis, material strength needs to be established using the information listed in the as-built plans.

Typically, $f'_c = \dfrac{f_c}{0.4}$ and therefore $f'_c = 3{,}000$ psi.

And for $f_s = 20{,}000$ psi and thus $f_y = 40{,}000$ psi (MBE Table 6B.5.2.3.1).

A. LFR Method

A.1. Dead Load Calculations

Self-weight of the interior girder = (27.5/12)(14/12)(0.15) = 0.401 kip/ft.

(4 × 4 in.) fillets between girder and slab = 2(1/2)(4/12)(4/12)(0.15) = 0.017 kip/ft.

Slab weight (based on tributary area) = (7.25)(6.5/12)(0.15) = 0.589 kip/ft.

Contribution from barrier rail (equally distributed among girders) = 2(0.25/6) = 0.083 kip/ft.

Thus, total uniform load on the interior girder = 1.090 kip/ft. (15.9 N/mm).

A.2. Live Load Calculations

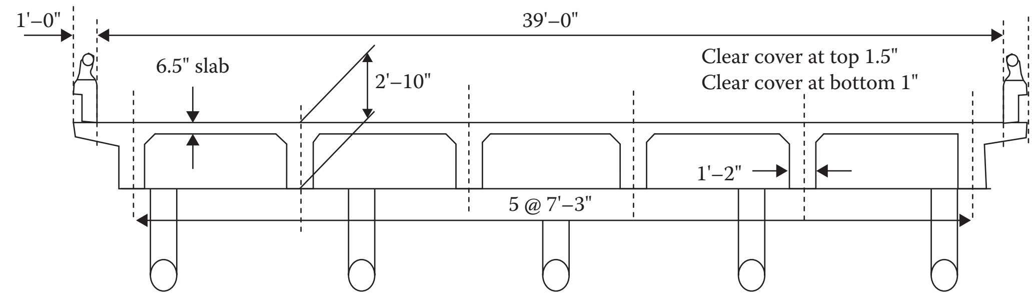
The travel width of this bridge is 39.0 ft. and therefore the number of live load wheels will be based on two or more traffic lanes.

From Table 3.23.1A of Standard Specification, LLDF for two or more traffic lanes for T-beams is given by S/6.0 wheel lines.

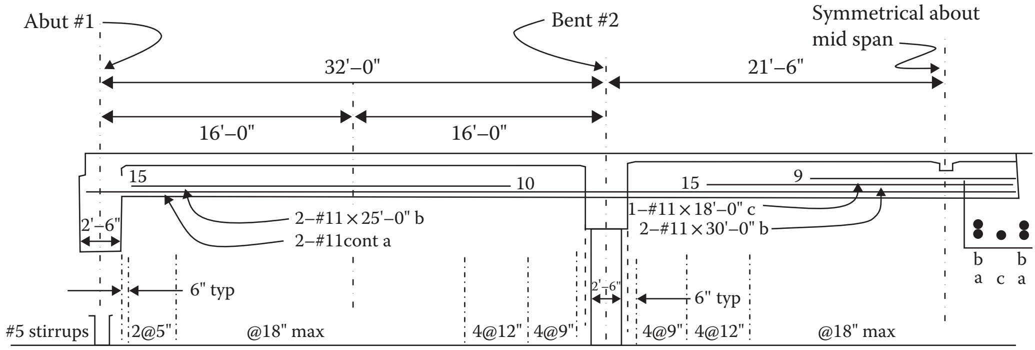
Number of live load wheel line/girder = 7.25/6.0 = 1.2083 wheels.

AASHTO Standard Impact factor [50/(125 + L)], where L is the effective span length (this will vary with the demand type).

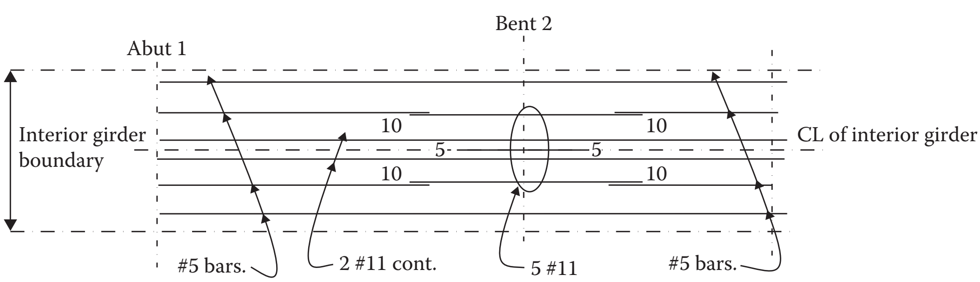
Analysis of three-span continuous superstructure framed into substructure cannot be performed by hand calculation. So, the analysis for dead and live loads is performed using simplified framed analysis



(a)



(b)



(c)

FIGURE 15.10 Details of three-span continuous RC T-beam bridge example: (a) Typical section, (b) bottom flexural and shear reinforcement, and (c) top flexural reinforcement detail of typical interior girder.

software. Analysis results are established at every 10th point span and listed in Tables 15.17 and 15.18. Since the bridge is symmetrical, demands for first half of the bridge are reported. In these tables, location identified as the span number and 10th point. The letter "F" designates the location at the face of the support and "*d*" designates the distance from face of the support. The demands at these locations are necessary to perform rating.

A.3. Moment Capacity Calculation

The effect of development length of each rebar is considered to establish the effective bar area at each analysis point. Articles 8.25.1, 8.25.2, and 8.25.3.1 of Standard Specification are used to establish the required development length of tensile rebar.

The depth below the top reinforcement is about 30 in. Since the depth below the rebars is greater than 12 in., per Article 8.25.2, basic development needs to be multiplied by a factor of 1.4. However, it is important to consider the construction sequence before considering this article. In most cases, the stem

TABLE 15.17 Interior Girder Moment Demands at 10th Points

		Moment Demand (kip-ft.)						
Location	Distance (ft.)	DL	HS20+	HS20−	P5+	P5−	P13+	P13−
Sp 1 – 0.0 pt.	0.00	0.0	4.0	0.0	0.0	0.0	0.0	0.0
Sp 1 – 0.04 pt.(F)	1.25	14.8	43.7	−6.4	52.1	−8.0	52.1	−7.0
Sp 1 – 0.10 pt.	3.20	34.5	100.0	−16.3	118.0	−20.5	118.0	−18.0
Sp 1 – 0.12 pt.(*d*)	3.83	39.9	115.2	−19.5	135.5	−24.6	135.5	−21.6
Sp 1 – 0.20 pt.	6.40	57.8	164.0	−32.6	187.9	−41.1	187.9	−36.0
Sp 1 – 0.30 pt.	9.60	69.9	195.5	−48.9	225.6	−61.6	225.6	−54.0
Sp 1 – 0.40 pt.	12.80	71.0	198.2	−65.2	237.1	−82.2	237.1	−72.0
Sp 1 – 0.50 pt.	16.00	60.8	185.4	−81.5	221.6	−102.7	221.6	−90.0
Sp 1 – 0.60 pt.	19.20	39.5	180.2	−97.8	208.7	−123.2	208.7	−108.0
Sp 1 – 0.70 pt.	22.40	7.0	148.4	−114.1	166.1	−143.8	166.1	−126.0
Sp 1 – 0.80 pt.	25.60	−36.6	91.2	−130.4	99.8	−164.0	99.8	−144.0
Sp 1 – 0.88 pt.(*d*)	28.17	−79.7	50.2	−143.5	31.5	−183.4	31.5	−196.8
Sp 1 – 0.90 pt.	28.80	−91.4	41.9	−146.7	32.2	−197.2	32.2	−219.3
Sp 1 – 0.96 pt.(F)	30.75	−130.2	28.6	−157.8	34.4	−240.2	34.4	−301.7
Sp 1 – 1.00 pt.	32.00	−157.3	29.7	−182.3	35.8	−268.1	35.8	−359.6
Sp 2 – 0.00 pt.	32.00	−159.1	32.7	−185.0	39.4	−283.1	39.4	−365.4
Sp 2 – 0.03 pt.(F)	33.25	−130.6	28.6	−158.8	34.4	−225.7	34.4	−309.7
Sp 2 – 0.09 pt.(*d*)	35.83	−77.3	34.0	−110.8	24.1	−136.0	24.1	−200.7
Sp 2 – 0.10 pt.	36.30	−68.4	36.7	−103.3	22.2	−127.2	22.2	−182.1
Sp 2 – 0.20 pt.	40.60	2.2	110.9	−80.2	118.6	−98.9	122.0	−98.9
Sp 2 – 0.30 pt.	44.90	52.5	181.1	−66.1	209.5	−81.6	200.4	−81.6
Sp 2 – 0.40 pt.	49.20	82.7	219.2	−52.1	256.0	−64.2	235.2	−64.2
Sp 2 – 0.50 pt.	53.50	92.8	227.2	−47.1	274.7	−46.9	246.2	−46.9

TABLE 15.18 Interior Girder Shear Demands at 10th Points

		Shear Demand (kip)						
Location	Distance (ft.)	DL	HS20+	HS20−	P5+	P5−	P13+	P13−
Sp 1 – 0.0 pt.	0.00	12.5	37.4	−5.1	44.1	−6.4	44.1	−5.6
Sp 1 – 0.04 pt. (F)	1.25	11.2	34.9	−5.1	41.7	−6.4	41.7	−5.6
Sp 1 – 0.10 pt.	3.20	9.0	31.3	−5.1	36.9	−6.4	36.9	−5.6
Sp 1 – 0.12 pt. (*d*)	3.83	8.4	30.1	−5.2	35.3	−6.4	35.3	−5.6
Sp 1 – 0.20 pt.	6.40	5.5	25.6	−8.4	29.4	−6.4	29.4	−6.1
Sp 1 – 0.30 pt.	9.60	2.1	20.4	−12.1	22.4	−10.7	22.4	−12.0
Sp 1 – 0.40 pt.	12.80	−1.4	15.5	−15.3	16.6	−15.1	16.6	−18.8
Sp 1 – 0.50 pt.	16.00	−4.9	10.6	−18.2	10.8	−21.2	10.8	−24.6
Sp 1 – 0.60 pt.	19.20	−8.4	7.7	−22.6	4.0	−26.9	4.0	−30.5
Sp 1 – 0.70 pt.	22.40	−11.9	5.3	−28.2	1.1	−32.0	1.1	−36.9
Sp 1 – 0.80 pt.	25.60	−15.4	3.2	−33.4	1.1	−40.2	1.1	−45.1
Sp 1 – 0.88 pt. (*d*)	28.17	−18.2	1.8	−37.2	1.1	−46.6	1.1	−51.1
Sp 1 – 0.90 pt.	28.80	−18.9	1.5	−38.1	1.1	−48.1	1.1	−52.5
Sp 1 – 0.96 pt. (F)	30.75	−21.0	0.9	−40.7	1.1	−52.4	1.1	−56.5
Sp 1 – 1.00 pt.	32.00	−22.4	0.9	−42.1	1.1	−54.4	1.1	−58.3
Sp 2 – 0.00 pt.	32.00	23.4	44.6	−3.3	57.6	−4.0	61.4	−4.0
Sp 2 – 0.03 pt. (F)	33.25	22.1	43.3	−3.3	56.1	−4.0	59.4	−4.0
Sp 2 – 0.09 pt. (*d*)	35.83	19.3	40.3	−3.3	51.6	−4.0	53.4	−4.0
Sp 2 – 0.10 pt.	36.30	18.8	39.5	−3.3	50.4	−4.0	52.3	−4.0
Sp 2 – 0.20 pt.	40.60	14.1	33.8	−4.3	42.3	−4.0	44.9	−4.0
Sp 2 – 0.30 pt.	44.90	9.4	27.3	−7.0	33.0	−4.5	35.9	−4.0
Sp 2 – 0.40 pt.	49.20	4.7	21.4	−10.9	24.2	−9.3	26.6	−8.2
Sp 2 – 0.50 pt.	53.50	0.0	15.2	−15.4	15.0	−15.6	16.5	−16.8

(web) is first poured up to the fillet (about 7" below deck), and once the concrete within stem is hardened, deck concrete is poured. In this situation, the depth of pour below top reinforcement is less than 12 in. In other words, if the concrete of bridge superstructure is poured in several sequences, the factor of 1.4 need not be included. Unfortunately, since the construction sequence is not known or not given, it is conservatively assumed that entire bridge is built in one sequence.

A.3.1. Establish Effective Area of Rebar at Analysis Points

If the distance from the bar termination point to the rating analysis point is less than the minimum development length established in Table 15.19, bars are "partially effective." Percentage of effectiveness is taken using the following equation:

$$\%\ \text{Effectiveness of bar} = \frac{\text{Distance from termination point to analysis point}}{\text{Required rebar development length}} \times 100\%$$

A.3.2. Establish Effective Slab Width

$$\text{Effective width of the deck } b_{\text{eff}} = \text{minimum of } \begin{cases} 12t_s + b_w = 12 \times 6.5 + 14 = 92 \text{ in.} \\ \dfrac{\text{Span}}{4} = 96 \text{ in. for Span 1 \& 3; 129 in. for Span 2} \\ \text{Spacing} = 87 \text{ in. (controls)} \end{cases}$$

A.3.3. Establish the Effective Depth of Rebars

Bars marked "a" and "c" are placed at the bottom layer and bars marked "b" is placed on top of bars "a."

The centroid bars "a" and "c" from bottom fiber = Clear cover + diameter of stirrup + $d_b/2$ = 1.5 + 5/8 + 1.41/2 = 2.83 in.

The centroid of "b" rebars from bottom fiber = 1.5 + 5/8 + 1.41 + 1.41/2 = 4.24 in.

So, centroid of "a," "b," and "c" bars together in span 2 = $\dfrac{3 \times 2.83 + 2 \times 4.24}{5} = 3.394$ in.

And centroid of "a" and "b" bars together in spans 1 and 3 = $\dfrac{2 \times 2.83 + 2 \times 4.24}{4} = 3.535$ in.

Therefore, centroid of bottom bars from top fiber in spans 1 and 3 = 34 – 3.535 in. = 30.465 in.

Centroid of bottom bars from top fiber in span 2 = 34 – 3.394 in. = 30.606 in.

Centroid of top bars from bottom fiber = 34 – (1.5 + 5/8 + 1.41/2) = 31.17 in.

A.3.4. Moment Capacity Calculation

Moment capacity is given by $\varphi M_n = \varphi A_s f_y \left(d - \dfrac{a}{2} \right)$, where $a = \dfrac{A_s f_y}{0.85 f'_c b_{\text{eff}}}$.

Rating evaluation has been performed at all the 10th points of the span; however, results from a few locations [(1) where rebars are partially developed, (2) face of supports, and (3) where maximum demand occurs] are listed. The area of bottom and top reinforcements and moment capacity at these locations are listed in Table 15.20.

TABLE 15.19 Required Development Lengths of Top and Bottom Reinforcements

Bar (Location)	Area (in.²)	Diameter d_b (in.)	Basic Development (in.)	Factor for Depth	Factor for Spacing ≥ 6 in.	Required Development Length
#11 (top bar) 20 ft.	1.56	1.41	45.60	1.4	0.80	51.07 in. = 4.26 ft.
#11 (top bar) 10 ft.	1.56	1.41	45.60	1.4	1.00	63.84 in. = 5.32 ft.
#11 (bottom)	1.56	1.41	45.60	1.0	1.00	45.60 in. = 3.80 ft.

TABLE 15.20 Area of Steel and Positive and Negative Moment Capacity at Critical Location

Location	A_{sb} (in.^{2})	d_{sb} (in.)	$\varphi M_{n\text{-pos}}$ (kip-ft.)	A_{st} (in.^{2})	d_{st} (in.)	$\varphi M_{n\text{-neg}}$ (kip-ft.)
Sp 1 – 0.40 pt.	6.240	30.47	559.80	3.120	31.17	275.4
Sp 1 – 0.80 pt.	3.448	31.04	317.80	5.763	31.17	483.1
Sp 1 – 0.96 pt. (F)	3.120	31.17	289.10	7.284	31.17	592.0
Sp 1 – 1.00 pt.	3.120	31.17	289.10	7.706	31.17	704.5[a]
Sp 2 – 0.00 pt.	3.120	31.17	289.10	7.706	31.17	704.5[a]
Sp 2 – 0.029 pt. (F)	3.120	31.17	289.10	7.284	31.17	592.0
Sp 2 – 0.2 pt.	4.844	30.67	439.30	4.148	31.17	359.0
Sp 2 – 0.3 pt.	6.404	30.48	574.50	3.120	31.17	275.4
Sp 2 – 0.5 pt.	7.800	30.61	699.70	3.120	31.17	275.4

[a] Moment capacity is based on rectangular shape of 87" wide × 34" bent cap.

A.3.5. Shear Capacity Calculation

Shear is checked at all the 10th points that are outside the "*d*" distance from support and at sections that are "*d*" distance from face of the support.

Shear capacity is then given by $\varphi V_n = \varphi\ (V_s + V_c)$, where φ is 0.85.

A.3.5.1. Shear Capacity Due to Concrete

Shear capacity due to concrete section V_c is given by

$$V_c = 2\sqrt{f_c'}\, b_w\, d \tag{15.16}$$

Or

$$V_c = \left[1.9\sqrt{f_c'} + 2500\rho_w\left(\frac{V_u d}{M_u}\right)\right] b_w\, d \tag{15.17}$$

In the preceding equations, d is the distance from compression fiber to the centroid of tensile rebar. When evaluating for shear capacity, shear demand and corresponding moment should be considered. The sign corresponding moment demand (positive or negative moment demand) will determine the tensile bar location. If the corresponding moment is positive moment, the bottom reinforcement becomes the tensile bar. On the contrary, if the corresponding moment is negative moment, top reinforcement becomes the tensile rebar. In this bridge example, depth d for positive and negative bending is 30.465 and 31.17 in., respectively. Since these values are approximately same, instead of identifying the tensile bar using corresponding moment demand, the smallest d (30.465 in.) is used to establish the capacity in this example. This would yield somewhat conservative results. Most commercially available software utilizes the shear and corresponding moment when establishing d and shear capacity.

Of the preceding two equations, Equation 15.17 will provide higher capacity. However, it is important to note that this equation requires the factored shear demand and corresponding factored moment demand. And the factored demand is a function of the rating factor [Factored demand = γ DL + (RF) $\gamma\beta$ (LL + I)] and therefore solving for rating factor using this equation becomes an iterative process.

Since Equation 15.17 requires iterative process to obtain rating factor, it is not used in this example. Instead, the simplified first equation (Equation 15.16) is used to establish the shear capacity due to concrete and listed in Table 15.21.

A.3.5.1. Shear Capacity Due to Shear Stirrups

The shear capacity due to shear reinforcement V_s is given by $A_v \dfrac{f_y d_s}{S}$ where S is the stirrup spacing. Typically, engineers use the largest spacing of stirrups at the analysis point when establishing the capacity due to stirrups.

Whenever the stirrup spacing S changes around an analysis point (see Figure 15.3), the number of legs that crosses the failure plane cannot be correctly established using the "S" value at analysis point A. As a result, average stirrup spacing found within "$d/2$" distance on either side of the analysis point is used to establish the shear capacity. The average stirrup spacing can be derived as $\left(\dfrac{S_1 S_2}{S_1 b + S_2 a}\right) d_v$, provided that the angle of shear failure angle is 45° (see Section 15.2.6.2 for more details). Established average shear bar spacing (S) and shear capacity due to shear reinforcement V_s are listed in Table 15.21.

A.4. Rating Calculations

Rating at an analysis point is typically given by the following equation:

$$\text{RF} = \frac{C - A_1 D}{A_2 \times (\text{LL} + I)} \tag{15.14}$$

where $A_1 = 1.3$ and $A_2 = 2.17$ and 1.3 for inventory rating and operating rating. $A_2 = 1.3$ for permit rating (MBE Article 6B.4.3). Using the rating equations, rating factor based on positive moment, negative moment and shear are established at all 10th points and face of support at listed in Table 15.22, Table 15.23, and Table 15.24 respectively.

TABLE 15.21 Shear Capacity at Several 10th Points

Location	V_c (kip)	S (in.)	V_s (kip)	φV_n (kip)
Sp 1 – 0.12 pt. (*d*)	46.88	17.52	43.27	76.63
Sp 1 – 0.2 pt.	46.722	18	41.97	75.39
Sp 1 – 0.3 pt.	46.722	18	41.97	75.39
Sp 1 – 0.4 pt.	46.722	18	41.97	75.39
Sp 1 – 0.5 pt.	46.722	18	41.97	75.39
Sp 1 – 0.6 pt.	46.722	18	41.97	75.39
Sp 1 – 0.7 pt.	46.751	16.63	45.46	78.38
Sp 1 – 0.8 pt.	47.597	12	64.14	94.98
Sp 1 – 0.88 pt. (*d*)	47.803	9.34	82.76	110.98
Sp 2 – 0.08 pt. (*d*)	47.803	9.34	82.76	110.98
Sp 2 – 0.1 pt.	47.803	9.8	78.88	107.68
Sp 2 – 0.2 pt.	47.033	14.07	54.06	85.93
Sp 2 – 0.3 pt.	46.749	18	42	75.44
Sp 2 – 0.4 pt.	46.938	18	42.17	75.74
Sp 2 – 0.5 pt.	46.938	18	42.17	75.74

TABLE 15.22 LFR Calculation Based on Positive Moment Demand

Location	$\varphi M_{n\text{-pos}}$ (kip-ft.)	DL_M (kip-ft.)	HS20 + I (kip-ft.)	Inventory RF	Operating RF	P5 + I (kip-ft.)	P5 RF	P13 + I (kip-ft.)	P13 RF
Sp 1 – 0.40 pt.	559.8	68.0	198.9	**1.09**	**1.82**	237.8	**1.52**	237.8	**1.52**
Sp 1 – 0.80 pt.	317.8	–41.1	92.5	1.79	2.98	101.5	2.72	101.5	2.72
Sp 1 – 0.96 pt. (F)	289.1	–130.8	29.2	6.63	11.06	35.1	9.20	35.1	9.20
Sp 1 – 1.00 pt.	*289.1*	*–162.0*	*30.5*	*6.82*	*11.38*	*36.8*	*9.43*	*36.8*	*9.43*
Sp 2 – 0.00 pt.	*289.1*	*–163.7*	*32.8*	*6.36*	*10.62*	*39.5*	*8.82*	*39.5*	*8.82*
Sp 2 – 0.029 pt. (F)	289.1	–130.1	28.0	6.90	11.52	33.7	9.57	33.7	9.57
Sp 2 – 0.2 pt.	439.3	3.4	112.2	1.79	2.98	120.2	2.78	123.5	2.71
Sp 2 – 0.3 pt.	574.5	56.8	182.4	1.26	2.11	211.1	1.82	201.6	1.91
Sp 2 – 0.5 pt.	699.7	104.2	228.4	1.14	1.90	276.2	1.57	247.2	1.76

TABLE 15.23 LFR Calculation Based on Negative Moment Demand

Location	φM_{neg} (kip-ft.)	DL_M (kip-ft.)	HS20 + I (kip-ft.)	Inventory RF	Operating RF	P5 + I (kip-ft.)	P5 RF	P13 + I (kip-ft.)	P13 RF
Sp 1 – 0.40 pt.	275.4	68.0	–66.2	2.39	3.99	–83.4	3.17	–73.0	3.62
Sp 1 – 0.80 pt.	483.1	–41.1	–132.5	1.49	2.49	–166.6	1.98	–146.0	2.26
Sp 1 – 0.96 pt. (F)	592.0	–130.8	–158.1	**1.23**	**2.05**	–238.2	**1.36**	–294.1	**1.10**
Sp 1 – 1.00 pt.	*704.5*	*–162.0*	*–182.9*	*1.25*	*2.08*	*–270.5*	*1.41*	*–360.9*	*1.05*
Sp 2 – 0.00 pt.	*704.5*	*–163.7*	*–184.9*	*1.23*	*2.05*	*–282.0*	*1.35*	*–365.3*	*1.04*
Sp 2 – 0.029 pt. (F)	592.0	–130.1	–154.8	1.26	2.10	–216.1	1.51	–300.9	1.08
Sp 2 – 0.2 pt.	359.0	3.4	–81.2	2.06	3.43	–100.2	2.78	–100.2	2.78
Sp 2 – 0.3 pt.	275.4	56.8	–67.0	2.28	3.81	–82.7	3.09	–82.7	3.09
Sp 2 – 0.5 pt.	275.4	104.2	–47.9	3.65	6.10	–47.6	6.13	–47.6	6.13

TABLE 15.24 LFR Calculation Based on Shear Demand

Location	φV_n (kip)	V_d (kip)	HS20 + I (kip)	Inventory RF	Operating RF	P5 + I (kip)	P5 RF	P13 + I (kip)	P13 RF
Sp 1 – 0.12 pt. (*d*)	76.63	8.1	30.1	1.01	1.69	35.4	1.44	35.4	1.44
Sp 1 – 0.2 pt.	75.39	5.3	25.7	1.23	2.05	29.4	1.79	29.4	1.79
Sp 1 – 0.3 pt.	75.39	1.9	20.4	1.65	2.75	22.5	2.50	22.5	2.50
Sp 1 – 0.4 pt.	75.39	–1.6	–15.4	2.20	3.67	16.7	3.55	–18.8	3.00
Sp 1 – 0.5 pt.	75.39	–5.1	–18.2	1.74	2.91	–21.2	2.50	–24.6	2.15
Sp 1 – 0.6 pt.	75.39	–8.5	–22.5	1.32	2.20	–26.9	1.84	–30.5	1.62
Sp 1 – 0.7 pt.	78.38	–12.0	–28.2	1.03	1.72	–31.9	1.51	–36.9	1.31
Sp 1 – 0.8 pt.	94.98	–15.4	–33.4	1.03	1.73	–40.2	1.43	–45.2	1.28
Sp 1 – 0.88 pt. (*d*)	110.98	–18.2	–37.2	1.08	1.81	–46.5	1.44	–51.1	1.31
Sp 2 – 0.08 pt. (*d*)	110.98	19.9	40.3	0.97	1.62	51.6	1.27	53.4	1.23
Sp 2 – 0.1 pt.	107.68	19.4	39.5	0.96	1.61	50.4	1.26	52.3	1.21
Sp 2 – 0.2 pt.	85.93	14.8	33.7	**0.91**	**1.52**	42.3	**1.21**	45.0	**1.14**
Sp 2 – 0.3 pt.	75.44	10.1	27.3	1.05	1.75	33.0	1.45	35.9	1.33
Sp 2 – 0.4 pt.	75.74	5.5	21.4	1.48	2.46	24.2	2.18	26.6	1.98
Sp 2 – 0.5 pt.	75.74	–0.9	–15.4	2.23	3.72	–15.6	3.67	–16.8	3.42

Please note that the rating based on negative moment demand over the centerline of bent is almost same as the rating established at the face of the bent cap. Therefore, the rating established over the centerline of the bent can be ignored.

A.5. Summary

Critical rating factor of the interior girder will then be 0.91 at inventory level and 1.52 at operating rating level for HS20 vehicle. The critical rating factor for routine permit P5 truck and special permit truck P13 will be 1.21 and 1.10, respectively.

B. LRFR Method

B.1. Dead Load Calculations

Dead load demand estimation of the interior girder is based on the tributary area of the girder and has been done in Section A.1.

B.2. Live Load Calculations

B.2.1. Moment LLDF ($LLDF_M$)

LLDF for concrete T-beams is given in Table 4.6.2.2.2.b-1 of the LRFD Specification.

One-lane loaded distribution factor $g_{1M} = 0.060 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$

Two or more lanes loaded distribution factor $g_{2M} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$

where $K_g = I + A \times e_g^2 = \frac{1}{12} \times 14 \times 27.5^3 + 14 \times 27.5 \times (27.5/2 + 6.5/2)^2 = 135{,}528$ in.^{4}

$$\text{So, } g_{1M} = 0.060 + \left(\frac{7.25}{14}\right)^{0.4}\left(\frac{7.25}{L}\right)^{0.3}\left(\frac{135{,}528}{12 \times L \times 6.5^3}\right)^{0.1}$$

$$\text{And, } g_{2M} = 0.075 + \left(\frac{7.25}{9.5}\right)^{0.6}\left(\frac{7.25}{L}\right)^{0.2}\left(\frac{135{,}528}{12 \times L \times 6.5^3}\right)^{0.1}$$

The span length (L) to be used in the preceding equations depends on the type of force effect considered. L is the length of the span that is being calculated for positive moment and is the average length of two adjacent spans for negative moment near interior supports. The span length L for different force types are established and listed in Table 15.25.

MBE requires that when evaluating bridges for “special permits,” single-lane distribution without the MPFs of 1.2 be used. The LLDF for the trucks are also established and listed in Table 15.25.

B.2.2. Shear LLDF (LLDF$_V$)

LLDF equations for concrete T-beams are given in Table 4.6.2.2.3.a-1 of the LRFD Specification.

One-lane loaded LLDF $g_{1V} = 0.36 + \frac{S}{25}$

Two or more lanes loaded distribution factor $g_{2V} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2$

Since the LLDF for shear depends only on girder spacing, the LLDF for girder along the entire span will remain the same.

$$g_{1V} = 0.36 + \frac{7.25}{25} = 0.65$$

$$g_{2V} = 0.2 + \frac{7.25}{12} - \left(\frac{7.25}{35}\right)^2 = 0.7613$$

Again, the MBE requires that when evaluating bridges for “special permits,” single-lane distribution without the MPFs of 1.2 be used. So the LLDF for P13 truck will be 0.65/1.2 = 0.5417.

TABLE 15.25 Live Load Distribution Factor along the Girder

Moment Type	Location	Span Length (ft.)	Single-Lane LLDF (g_{1M})	Two or More Lanes Loaded LLDF for HL93 and P5 Truck (g_{2M})	Single-Lane LLDF for P13 Truck (g_{1M}/1.2)
Positive moment	Spans 1 & 3	32.0	0.565	0.723	0.565
Negative moment	Over the bents 2 & 3 (up to inflection points)	37.5	0.534	0.693	0.534
Positive moment	Span 2	43.0	0.509	0.668	0.509
Negative moment	In spans 1 & 3 (predominantly positive moment region)	32.0	0.565	0.723	0.565
Negative moment	In span 2 (predominantly positive moment region)	43.0	0.509	0.668	0.509

B.2.3. Live Load Demand at All Important Points

The live load demands are obtained using two-dimensional analysis software. Since the bridge is symmetrical about midspan of the span, results are listed at every 10th point up to midpoint of the span in Tables 15.26 and 15.27.

B.3. Capacity Calculations

Strengths of concrete and rebars were established earlier and will be used in the following calculations. It is listed here for reference: $f'_c = 3{,}000$ psi; $f_y = 40{,}000$ psi.

B.3.1. Moment Capacity at All 10th Points

The positive and negative moment capacity of the girder along the girder is established using the following expression:

$$\varphi M_n = \varphi A_s f_y \left(d - \frac{a}{2} \right)$$

where $a = \dfrac{A_s f_y}{0.85 f'_c b_{eff}}$ and $c = \dfrac{a}{\beta}$

B.3.2. Effective Width of the Deck b_{eff}

According to Article 4.6.2.6 of 6th Edition of LRFD Specification, effective flange width can be taken as the tributary width.

Therefore, b_{eff} = girder spacing = 7.25 ft. = 87 in.

TABLE 15.26 Critical Live Load Moment Demands on the Interior Girder

Location	Distance from Support 1 (ft.)	DL_M (kip-ft.)	HL93 + I (Pos) (kip-ft.)	HL93 + I (Neg) (kip-ft.)	P5 + I (Pos) (kip-ft.)	P5 + I (Neg) (kip-ft.)	P13 + I (Pos) (kip-ft.)	P13 + I (Neg) (kip-ft.)
Sp 1 – 0.04 pt. (F)	1.3	14.2	61.1	–9.7	63.5	–10.0	49.4	–6.8
Sp 1 – 0.10 pt.	3.2	33.0	142.2	–24.7	143.8	–25.7	112.5	–17.3
Sp 1 – 0.12 pt. (*d*)	3.8	38.3	165.2	–29.6	165.0	–30.7	129.2	–20.8
Sp 1 – 0.20 pt.	6.4	55.2	242.4	–49.5	228.7	–51.3	179.3	–34.7
Sp 1 – 0.30 pt.	9.6	66.5	302.5	–74.3	275.6	–77.0	215.5	–52.0
Sp 1 – 0.40 pt.	12.8	66.9	325.1	–99.1	289.3	–102.7	226.5	–69.4
Sp 1 – 0.50 pt.	16.0	56.3	319.8	–123.8	269.8	–128.3	211.7	–86.8
Sp 1 – 0.60 pt.	19.2	34.9	290.2	–148.5	254.1	–154.0	199.4	–104.1
Sp 1 – 0.70 pt.	22.4	2.6	231.6	–173.4	203.6	–179.6	159.0	–121.4
Sp 1 – 0.80 pt.	25.6	–40.5	143.1	–189.9	117.9	–196.8	92.5	–133.0
Sp 1 – 0.88 pt. (*d*)	28.2	–83.1	72.3	–213.6	38.3	–216.5	29.9	–180.6
Sp 1 – 0.90 pt.	28.8	–94.6	58.2	–220.9	39.1	–232.6	30.6	–200.2
Sp 1 – 0.96 pt. (F)	30.8	–133.0	41.2	–246.5	41.8	–283.4	32.7	–272.9
Sp 1 – 1.00 pt.	32.0	–159.6	42.5	–285.8	43.5	–316.4	34.0	–325.6
Sp 2 – 0.00 pt.	32.0	–161.3	45.6	–289.3	46.7	–329.7	35.6	–329.6
Sp 2 – 0.03 pt. (F)	33.3	–132.6	40.2	–246.2	40.7	–262.4	31.0	–278.9
Sp 2 – 0.09 pt. (*d*)	35.8	–78.3	61.5	–169.3	28.4	–161.4	21.6	–179.9
Sp 2 – 0.10 pt.	36.3	–69.2	68.0	–157.4	26.2	–151.2	20.0	–163.0
Sp 2 – 0.20 pt.	40.6	3.4	170.4	–115.0	142.8	–117.1	112.0	–90.2
Sp 2 – 0.30 pt.	44.9	56.0	254.8	–93.4	241.6	–93.1	176.3	–71.7
Sp 2 – 0.40 pt.	49.2	89.2	310.0	–77.3	291.1	–73.4	205.5	–56.6
Sp 2 – 0.50 pt.	53.5	102.7	326.3	–61.2	313.1	–53.7	213.6	–41.4

Note: (F) denotes the face of support; (*d*) denotes *d* distance away from the face of support.

TABLE 15.27 Critical Live Load Shear Demands on the Interior Girder

Location	Distance from Support 1 (ft.)	DL_V (kip)	HL93 + I (Pos) (kip)	HL93 + I (Neg) (kip)	P5 + I (Pos) (kip)	P5 + I (Neg) (kip)	P13 + I (Pos) (kip)	P13 + I (Neg) (kip)
Sp 1 – 0.04 pt. (F)	1.3	14.2	61.1	–9.7	63.5	–10.0	49.4	–6.8
Sp 1 – 0.10 pt.	3.2	33.0	142.2	–24.7	143.8	–25.7	112.5	–17.3
Sp 1 – 0.12 pt. (*d*)	3.8	38.3	165.2	–29.6	165.0	–30.7	129.2	–20.8
Sp 1 – 0.20 pt.	6.4	55.2	242.4	–49.5	228.7	–51.3	179.3	–34.7
Sp 1 – 0.30 pt.	9.6	66.5	302.5	–74.3	275.6	–77.0	215.5	–52.0
Sp 1 – 0.40 pt.	12.8	66.9	325.1	–99.1	289.3	–102.7	226.5	–69.4
Sp 1 – 0.50 pt.	16.0	56.3	319.8	–123.8	269.8	–128.3	211.7	–86.8
Sp 1 – 0.60 pt.	19.2	34.9	290.2	–148.5	254.1	–154.0	199.4	–104.1
Sp 1 – 0.70 pt.	22.4	2.6	231.6	–173.4	203.6	–179.6	159.0	–121.4
Sp 1 – 0.80 pt.	25.6	–40.5	143.1	–189.9	117.9	–196.8	92.5	–133.0
Sp 1 – 0.88 pt. (*d*)	28.2	–83.1	72.3	–213.6	38.3	–216.5	29.9	–180.6
Sp 1 – 0.90 pt.	28.8	–94.6	58.2	–220.9	39.1	–232.6	30.6	–200.2
Sp 1 – 0.96 pt. (F)	30.8	–133.0	41.2	–246.5	41.8	–283.4	32.7	–272.9
Sp 1 – 1.00 pt.	32.0	–159.6	42.5	–285.8	43.5	–316.4	34.0	–325.6
Sp 2 – 0.00 pt.	32.0	–161.3	45.6	–289.3	46.7	–329.7	35.6	–329.6
Sp 2 – 0.03 pt. (F)	33.3	–132.6	40.2	–246.2	40.7	–262.4	31.0	–278.9
Sp 2 – 0.09 pt. (*d*)	35.8	–78.3	61.5	–169.3	28.4	–161.4	21.6	–179.9
Sp 2 – 0.10 pt.	36.3	–69.2	68.0	–157.4	26.2	–151.2	20.0	–163.0
Sp 2 – 0.20 pt.	40.6	3.4	170.4	–115.0	142.8	–117.1	112.0	–90.2
Sp 2 – 0.30 pt.	44.9	56.0	254.8	–93.4	241.6	–93.1	176.3	–71.7
Sp 2 – 0.40 pt.	49.2	89.2	310.0	–77.3	291.1	–73.4	205.5	–56.6
Sp 2 – 0.50 pt.	53.5	102.7	326.3	–61.2	313.1	–53.7	213.6	–41.4

Note: (F) denotes the face of support; (*d*) denotes *d* distance away from the face of support.

TABLE 15.28 Required Development Lengths of Top and Bottom Reinforcements

Bar (Location)	Area (in.^{2})	Diameter d_b (in.)	Basic Development (in.)	Factor for Depth	Factor for Spacing ≥ 6 in.	Required Development Length
#11 (top bar) 20 ft.	1.56	1.41	45.03	1.4	0.80	50.44 in. = 4.20 ft.
#11 (top bar) 10 ft.	1.56	1.41	45.03	1.4	1.00	63.04 in. = 5.25 ft.
#11 (bottom)	1.56	1.41	45.03	1.0	1.00	45.03 in. = 3.75 ft.

B.3.3. Effect of Rebar Development Length

As stated in the preceding sections A.3.1 and A.3.4, considering the effect of development length of each rebar, the effective bar area at every 10th point is established and listed in Tables 15.28 and 15.29, respectively.

The basic development length of #11 bar $= \dfrac{1.25A_b \times f_y}{\sqrt{f_c'}}$, but not less than $0.4d_b f_y$.

So, $L_{Dev} = \dfrac{1.25A_b \times f_y}{\sqrt{f_c'}} = \dfrac{1.25 \times 1.56 \times 40}{\sqrt{3}} = 45.03\ \text{in.} = 3.75\ \text{ft.} > 0.4d_b f_y = 1.88\ \text{ft.}$

B.3.4. Moment Reduction Factor φ

Within LRFD specification, the moment reduction factor varies with the tensile strain in the tensile rebar. The strain at the centroid of the tension bars can be established by the following equation (Article C5.7.2.1-1 and Equation 5.7.3.1.2-4)

$$\varepsilon_t = \frac{0.003(d_s - c)}{c}, \text{where } c = \frac{A_s f_y}{0.85\beta_1 f_c' b}$$

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TABLE 15.29 Effective Area of Tensile Steel and Positive and Negative Moment Capacity at Critical Locations Using the LRFR analysis

Location	A_{sb} (in.^{2})	d_{sb} (in.)	$\varphi M_{n\text{-}pos}$ (kip-ft.)	A_{st} (in.^{2})	d_{st} (in.)	$\varphi M_{n\text{-}neg}$ (kip-ft.)
Sp 1 – 0.30 pt.	6.240	30.47	559.80	3.120	31.17	275.4
Sp 1 – 0.40 pt.	6.240	30.47	559.80	3.120	31.17	275.4
Sp 1 – 0.50 pt.	6.240	30.47	559.80	3.120	31.17	275.4
Sp 1 – 0.60 pt.	6.240	30.47	559.80	3.120	31.17	275.4
Sp 1 – 0.70 pt.	6.115	30.48	549.00	3.417	31.17	299.9
Sp 1 – 0.96 pt. (F)	3.120	31.17	289.10	7.354	31.17	596.8
Sp 1 – 1.00 pt.	3.120	31.17	289.10	7.726	31.17	706.3[a]
Sp 2 – 0.00 pt.	3.120	31.17	289.10	7.726	31.17	706.3[a]
Sp 2 – 0.029 pt. (F)	3.120	31.17	289.10	7.354	31.17	596.8
Sp 2 – 0.2 pt.	4.867	30.66	441.30	4.160	31.17	359.9
Sp 2 – 0.3 pt.	6.406	30.48	574.70	3.120	31.17	275.4
Sp 2 – 0.5 pt.	7.800	30.61	699.70	3.120	31.17	275.4

Note: (F) denotes the face of the bent cap (or support).
[a] Moment capacity is based on rectangular shape of 87" wide × 34" bent cap.

For strain $\varepsilon_t \geq 0.005$, the moment reduction factor φ is 0.90; for $0.005 \geq \varepsilon_t \geq 0.002$, moment reduction factor φ is $0.65 + 0.15\left(\frac{d_t}{c} - 1\right)$; and for strain $\varepsilon_t \leq 0.002$, moment reduction factor φ is 0.75.

Positive and Negative moment capacities at all the 10th points within first half of the girder is established and listed in Table 15.29. Table 15.29 also lists the effective area and depth used to establish the positive and negative moment capacities.

B.3.5. Shear Capacity

As stated in Example 15.3.1, there are four alternative methods available within LRFD Specifications (Article 5.8.3.4.1, Article 5.8.3.4.3, Article 5.8.3.4.2 of 4th LRFD Edition, and Article 5.8.3.4.2 of 6th LRFD Edition) to establish the shear capacity. In this example, general procedure listed (Article 5.8.3.4.2) in the 6th Edition of the LRFD Specification is used to establish the shear capacity.

Shear capacity of a reinforced concrete is smallest of $\begin{cases} V_n = V_c + V_S \\ V_n = 0.25 f_c' b_v d_v \end{cases}$, where

$V_s = \dfrac{A_v f_y d_v \cot\theta}{s}$ due to vertical shear rebars, $V_c = 0.0316\,\beta\sqrt{f_c'} b_v d_v$,

$$\beta = \frac{4.8}{1 + 750\varepsilon_s} \quad \text{and} \quad \theta = 29 + 3500\,\varepsilon_s$$

The longitudinal strain for reinforced concrete girders at tension reinforcement can be taken as the following equation (derived from Equation 5.8.3.4.2 of 6th Edition of LRFD Specification, by removing variables that are not applicable.)

$$\varepsilon_s = \frac{\left|\dfrac{M_u}{d_v}\right| + V_u}{E_s A_s} \quad \text{where} \left|\frac{M_u}{d_v}\right| \geq V_u$$

The effective shear depth d_v for establishing shear that has corresponding *positive moment* is given (LRFD Article 5.8.2.9) by

$$\text{Maximum of} \begin{cases} d_e - \dfrac{a}{2} \approx 30.45 - 1.125/2 = 29.89 \text{ in. (controls)} \\ 0.9 d_e = 0.9 \times 30.45 = 27.41 \text{ in.} \\ 0.72\text{h} = 0.72 \times 34 = 24.48 \text{ in.} \end{cases}$$

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The effective shear depth d_v for establishing shear that has corresponding negative moment is given (LRFD Article 5.8.2.9) by

$$\text{Maximum of}\begin{cases} d_e - \dfrac{a}{2} \approx 31.17 - 7.726/2 = 27.31 \text{ in. (controls)} \\ 0.9d_e = 0.9 \times 31.17 = 28.05 \text{ in.} \\ 0.72\text{h} = 0.72 \times 34 = 24.48 \text{ in.} \end{cases}$$

So, effective shear depth d_v will be taken as 29.89 in. where moment demand corresponding to shear creates tension in bottom reinforcement and 27.31 in. where moment demand corresponding to shear creates tension in top reinforcement.

The angle of inclination of the shear crack θ and factor β that indicates the ability of diagonal crack to become tension crack depends on the longitudinal strain (ε_x) at mid-depth and shear stress (v_u) on the concrete.

Whenever the stirrup spacing S changes around an analysis point (see Figure 15.3), the number of shear reinforcements that crosses the shear failure plane cannot be correctly established using the "S" value at the analysis point A. As a result, average stirrup spacing found within "$d/2$" distance on either side of the analysis point is used to establish the shear capacity.

The average stirrup spacing is given as $\left(\dfrac{S_1 S_2}{S_1 b + S_2 a}\right) d\cot(\theta)$, where θ is angle of shear failure angle (see Section 15.2.6.2).

The rating analysis needs to be performed considering two scenarios: (1) maximum shear demand and corresponding moment and (2) maximum moment and corresponding shear. However, in this example, the analysis is performed using the maximum moment and shear demands at the analysis point. This approximate analysis typically yields conservative results.

Typically, during rating analysis, the factored demand is a function of the rating factor of the live load vehicle [L_u = 1.25 DL + 1.5 DW + γ (RF) (LL + I)], the shear capacity varies with the factored demand on the girder and rating factor depends on the capacity of the member. Since rating factor is not known, the solution can only be performed by iteration procedure. The shear capacity is first established using the factored demand using an "assumed" rating factor of 1.00. Using the established shear capacity, the rating factor is reestablished using the Equation 15.15. If the estimated rating factor does not match the assumed rating factor, this process will be repeated until assumed and estimated rating factor matches.

After trial and error method, the assumed rating factor and calculated rating factor matched. The shear capacity, assumed rating factor, angle of diagonal crack θ, and β are listed in Tables 15.32 through 15.35 later in the chapter.

B.4. Rating Calculations

$$\text{RF} = \frac{C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_P P}{\gamma_L\, L(1+I)} \tag{15.15}$$

According to MBE, γ_{DC} is 1.25. The live load factors are established in Section 15.3.1 and they are 1.75, 1.35, 1.58, and 1.50 for inventory rating, operating rating, routine permit truck, and special permit truck, respectively. By substituting these values and appropriate load effect values, the moment and shear rating are estimated. The calculations and results are given in Tables 15.30 through 15.35.

B.5. Summary

Critical rating of the interior girder will then be 0.84 at inventory level and 1.08 at operating rating level for HL93 vehicle. The permit rating for routine P5 truck and special trip P13 truck are 1.04 and 1.24, respectively.

TABLE 15.30 LRFR Calculation Based on Positive Moment Demand

Location	$M_{cap-pos}$ (kip-ft.)	DL (kip-ft.)	HL93 + I (kip-ft.)	Inventory RF	Operating RF	P5 + I (kip-ft.)	P5 RF	P13 + I (kip-ft.)	P13 RF
Sp 1 – 0.30 pt.	559.8	66.5	302.5	0.90	1.17	275.6	1.09	179.5	1.77
Sp 1 – 0.40 pt.	559.8	66.9	325.1	**0.84**	**1.08**	289.3	**1.04**	188.7	**1.68**
Sp 1 – 0.50 pt.	559.8	56.3	319.8	0.87	1.13	269.8	1.15	176.4	1.85
Sp 1 – 0.60 pt.	559.8	34.9	290.2	1.02	1.32	254.1	1.29	166.2	2.07
Sp 1 – 0.70 pt.	549.0	2.6	231.6	1.35	1.75	203.6	1.70	132.5	2.75
Sp 1 – 0.96 pt. (F)	289.1	–133.0	41.2	5.39	6.99	41.8	5.95	27.2	9.53
Sp 1 – 1.00 pt.	289.1	–159.6	42.5	5.50	7.13	43.5	5.56	28.3	9.63
Sp 2 – 0.00 pt.	289.1	–161.3	45.6	5.14	6.66	46.7	6.04	29.7	9.20
Sp 2 – 0.029 pt. (F)	289.1	–132.6	40.2	5.52	7.16	40.7	7.75	25.8	10.04
Sp 2 – 0.2 pt.	441.3	3.4	170.4	1.47	1.90	142.8	1.94	93.3	3.12
Sp 2 – 0.3 pt.	574.7	56.0	254.8	1.13	1.47	241.6	1.94	146.9	2.29
Sp 2 – 0.5 pt.	699.7	102.7	326.3	1.00	1.30	313.1	1.15	178.0	2.14

TABLE 15.31 LRFR Calculation Based on Negative Moment Demand

Location	$M_{cap-neg}$ (kip-ft.)	DL (kip-ft.)	HL93 + I (kip-ft.)	Inventory RF	Operating RF	P5 + I (kip-ft.)	P5 RF	P13 + I (kip-ft.)	P13 RF
Sp 1 – 0.30 pt.	275.4	66.5	–74.3	2.50	3.24	–77.0	2.67	–43.3	5.01
Sp 1 – 0.40 pt.	275.4	66.9	–99.1	1.88	2.43	–102.7	2.01	–57.8	3.76
Sp 1 – 0.50 pt.	275.4	56.3	–123.8	1.47	1.90	–128.3	1.57	–72.3	2.93
Sp 1 – 0.60 pt.	275.4	34.9	–148.5	1.16	1.50	–154.0	1.24	–86.8	2.32
Sp 1 – 0.70 pt.	299.9	2.6	–173.4	**0.99**	**1.29**	–179.6	1.06	–101.2	1.99
Sp 1 – 0.96 pt. (F)	596.8	-133.0	–246.5	1.00	1.29	–283.4	0.96	–227.4	1.26
Sp 1 – 1.00 pt.	*706.3*[a]	*–159.6*	*–285.8*	*1.01*	*1.31*	*–316.4*	*0.96*	*–271.3*	*1.26*
Sp 2 – 0.00 pt.	*706.3*[a]	*–161.3*	*–289.3*	*1.00*	*1.29*	*–329.7*	*1.01*	*–274.6*	*1.25*
Sp 2 – 0.029 pt. (F)	596.8	–132.6	–246.2	1.00	1.30	–262.4	**1.04**	–232.4	**1.24**
Sp 2 – 0.2 pt.	359.9	3.4	–115.0	1.80	2.33	–117.1	1.96	–75.2	3.21
Sp 2 – 0.3 pt.	533.1	56.0	–93.4	1.94	2.52	–93.1	2.16	–59.8	3.54
Sp 2 – 0.5 pt.	275.4	102.7	–61.2	3.29	4.27	–53.7	4.15	–34.5	6.81

Note: (F) denotes the face of the bent cap (or support).
[a] Moment capacity is based on rectangular shape of 87" wide × 34" bent cap.

TABLE 15.32 LRFR Inventory Rating Calculation Based on HL93 Shear

Location	S_{ave} (in.)	β	θ	V_c (kip)	V_s (kip)	φV_n (kip)	V_d (kip)	HL93 + I (kip)	Inventory RF
Sp 1 – 0.12 pt. (*d*)	17.52	2.18	34.60	50.23	61.70	100.74	8.0	44.8	1.16
Sp 1 – 0.2 pt.	18.00	2.00	35.52	45.83	57.72	93.19	5.2	38.9	1.27
Sp 1 – 0.8 pt.	12.00	2.08	35.12	45.70	84.26	116.97	–15.2	–50.0	1.12
Sp 1 – 0.88 pt. (*d*)	9.34	1.99	35.57	43.73	106.47	135.18	–17.9	–56.1	1.15
Sp 2 – 0.08 pt. (*d*)	9.34	2.14	34.78	47.02	109.64	140.99	19.6	61.4	1.08
Sp 2 – 0.1 pt.	9.80	2.18	34.59	47.90	105.23	137.82	19.1	60.0	1.08
Sp 2 – 0.2 pt.	14.07	2.11	34.94	48.87	76.25	112.61	14.6	50.8	**1.06**

Note: (*d*) denotes *d* distance away from the face of support.

15.3.5 Three-Span Continuous Reinforced Concrete Flat Slab Bridge

Given: A three-span continuous, longitudinally reinforced, 16.25-in. (420-mm)-thick concrete slab superstructure is supported by five 16.0-in. (406-mm)-diameter reinforced concrete column bents and abutments. The span lengths of each span is 30 ft. (9.1 m), and overall width of the bridge is 34 ft.

TABLE 15.33 LRFR Operating Rating Calculation Based on HL93 Shear

Location	S_{ave} (in.)	β	θ	V_c (kip)	V_s (kip)	φV_n (kip)	V_d (kip)	HL93 + I (kip)	Operating RF
Sp 1 – 0.12 pt. (*d*)	17.52	2.19	34.58	50.46	61.75	100.99	8.0	44.8	1.50
Sp 1 – 0.2 pt.	18.00	2.00	35.53	45.83	57.69	93.17	5.2	38.9	1.65
Sp 1 – 0.8 pt.	12.00	2.08	35.11	45.70	84.29	117.00	–15.2	–50.0	1.45
Sp 1 – 0.88 pt. (*d*)	9.34	1.99	35.57	43.73	106.47	135.18	–17.9	–56.1	1.49
Sp 2 – 0.08 pt. (*d*)	9.34	2.14	34.78	47.02	109.64	140.99	19.6	61.4	1.40
Sp 2 – 0.1 pt.	9.80	2.18	34.61	47.90	105.16	137.75	19.1	60.0	1.40
Sp 2 – 0.2 pt.	14.07	2.11	34.97	48.87	76.17	112.53	14.6	50.8	**1.37**

Note: (*d*) denotes *d* distance away from the face of support.

TABLE 15.34 LRFR Permit Rating Calculation Based on Routine P5 Permit Truck Shear

Location	S_{ave} (in.)	β	θ	V_c (kip)	V_s (kip)	φV_n (kip)	V_d (kip)	P5+ I (kip)	P5 RF
Sp 1 – 0.12 pt. (*d*)	17.52	2.18	34.60	50.23	61.70	100.74	8.0	45.4	1.28
Sp 1 – 0.2 pt.	18.00	2.04	35.32	46.74	58.14	94.40	5.2	37.7	1.43
Sp 1 – 0.8 pt.	12.00	2.06	35.23	45.27	83.92	116.27	–15.2	–50.9	1.23
Sp 1 – 0.88 pt. (*d*)	9.34	1.97	35.68	43.29	106.04	134.40	–17.9	–59.1	1.26
Sp 2 – 0.08 pt. (*d*)	9.34	2.13	34.84	46.80	109.39	140.58	19.6	66.2	1.20
Sp 2 – 0.1 pt.	9.80	2.17	34.66	47.68	104.96	137.38	19.1	64.6	1.20
Sp 2 – 0.2 pt.	14.07	2.12	34.91	49.10	76.34	112.90	14.6	54.1	**1.18**

Note: (*d*) denotes *d* distance away from the face of support.

TABLE 15.35 LRFR Permit Rating Calculation Based on Special P13 Permit Truck Shear

Location	S_{ave} (in.)	β	θ	V_c (kip)	V_s (kip)	φV_n (kip)	V_d (kip)	P13 + I (kip)	P13 RF
Sp 1 – 0.12 pt. (*d*)	17.52	2.32	34.00	53.46	63.11	104.91	8.0	32.9	1.41
Sp 1 – 0.2 pt.	18.00	2.35	33.87	53.85	61.38	103.70	5.2	27.3	1.66
Sp 1 – 0.8 pt.	12.00	2.36	33.81	51.86	88.49	126.32	–15.2	–41.3	1.43
Sp 1 – 0.88 pt. (*d*)	9.34	2.21	34.46	48.56	110.95	143.56	–17.9	–46.8	1.44
Sp 2 – 0.08 pt. (*d*)	9.34	2.25	34.27	49.44	111.75	145.07	19.6	50.3	1.31
Sp 2 – 0.1 pt.	9.80	2.31	34.04	50.76	107.43	142.37	19.1	48.9	1.32
Sp 2 – 0.2 pt.	14.07	2.31	34.02	53.50	78.92	119.18	14.6	41.7	**1.32**

(10.36 m) and has been carrying two lanes of traffic. Typical cross section and plan views that show the top and bottom longitudinal rebar layout are shown in Figure 15.11. General notes given in the as-built plan indicate that $f_c = 1{,}200$ psi (8.3 MPa) and $f_s = 18{,}000$ psi (124.1 MPa). Assume the weight of each barrier rail as 420 lb/ft. (6.6 N/mm). Clear cover distance for top and bottom reinforcements is 2.00 in.

Requirement: Assuming no deterioration of materials occurred, determine the critical rating factor of the bridge for the live load vehicles listed in Section 15.3.1 using (1) LFR method and (2) LRFR method.

Solution

This example is an extension of Section 15.3.3. In this example, because of continuity over the bents, the slab will experience both positive and negative moment demands. Most of the analysis concept is the same and as a result only brief references are made to the specification and equations.

As in Section 15.3.3, 12-ft.-wide reinforced concrete slab is used to rate the bridge using both LFR and LRFR methods. To illustrate the rating analysis, detailed calculations are performed at 7/10th point of span 1 first. All the necessary steps are followed at this location. Later, the rating is performed at every 10th point of the spans.

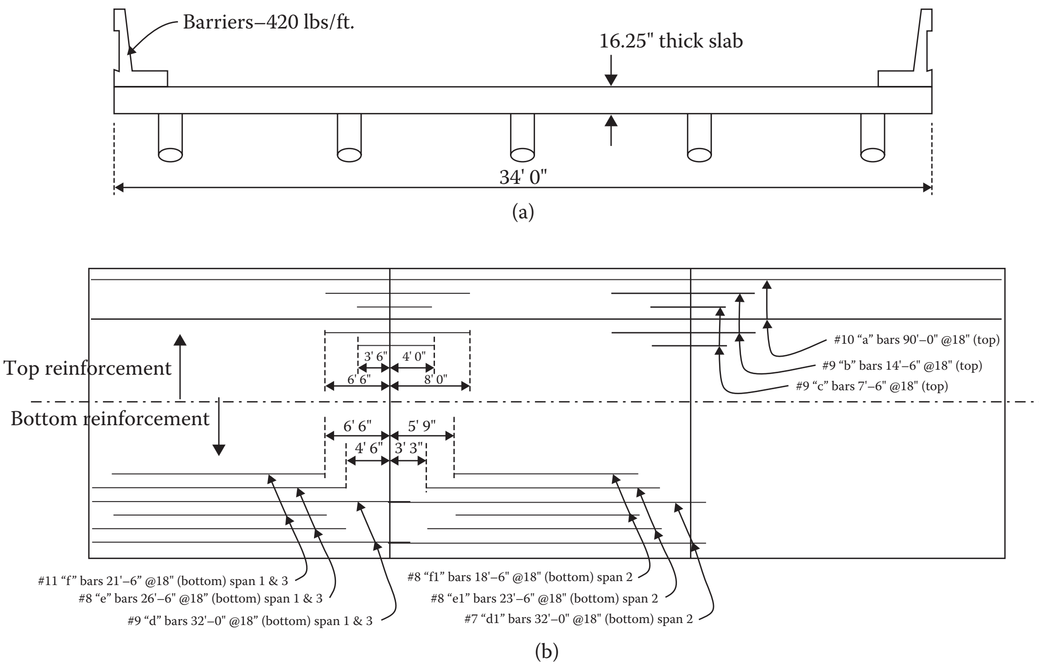

FIGURE 15.11 Details of three-span continuous RC slab bridge example: (a) Typical section and (b) reinforcement layout of 34-ft.-wide RC slab.

A. LFR Method

A.1. Dead Load Calculation

Self-weight of 12-ft.-wide slab = (12.0)(16.5/12)(0.15) = 2.475 kip/ft.

Barrier weights are conservatively distributed to entire slab equally and therefore the equivalent barrier weight per 12.0-ft. strip slab = (2)(0.420)(12)/(34) = 0.296 kip/ft.

So, the total uniform dead load = 2.475 + 0.296 = 2.771 kip/ft.

A.2. Live Load Calculation

From Article 3.23.3 of Standard Specification, the effective slab width that carries one wheel line is 4 + 0.06S, but limited to 7.0 ft.

$E = 4 + (0.06)(30) = 5.8$ ft. (< 7 ft.)

The number of live load wheels carried by 12-ft. slab = 12/5.8 = 2.069 wheel lines.

So, the LLDF = 2.069 wheel lines.

A.3. Live Load and Dead Load Analysis

Analysis of a three-span continuous superstructure that is framed into substructure cannot be performed by hand calculation. As a result, the analysis for dead and live loads is performed using simple framed analysis software. Since the bridge is symmetrical about midspan of span 2, results are listed at every 10th point of span 1 and up to midpoint of span 2 in Table 15.36.

A.4. Rating Calculation

Rating of bridge typically is done using the following equation:

$$\mathrm{RF} = \frac{C - A_1 D}{A_2 \times (\mathrm{LL} + I)} \tag{15.14}$$

The values for dead and live loads have already been established. Next step in establishing the rating factor is the capacity of the member. Here in this example, rating is performed *only* at every 10th point of the span. However, to illustrate the details of the calculations, all the steps performed to obtain the rating at bent 2 location and 7/10th of span 1 are documented.

TABLE 15.36 Dead and Live Load Moment Demand on 12-ft.-Wide Slab Strip

		Maximum Moment (kip-ft.)							
Location	Distance from Support 1 (ft.)	DL_M	HS + I (Pos)	HS + I (Neg)	P5 + I(Pos)	P5 + I(Neg)	P13 + I(Pos)	P13 + I (Neg)
Sp 1 – 0.0 pt.	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Sp 1 – 0.1 pt.	3.0	87.3	155.9	–14.9	179.4	–15.5	179.4	–12.9
Sp 1 – 0.2 pt.	6.0	149.6	253.1	–29.9	283.8	–31.0	283.8	–25.8
Sp 1 – 0.3 pt.	9.0	187.1	299.6	–44.8	335.5	–46.6	335.5	–38.7
Sp 1 – 0.4 pt.	12.0	199.5	301.6	–59.8	352.4	–62.1	352.4	–51.6
Sp 1 – 0.5 pt.	15.0	187.0	279.5	–74.7	332.4	–77.6	332.4	–64.5
Sp 1 – 0.6 pt.	18.0	149.6	271.2	–89.7	302.7	–93.1	302.7	–77.4
Sp 1 – 0.7 pt.	21.0	87.2	221.6	–104.6	240.3	–108.7	240.3	–90.4
Sp 1 – 0.8 pt.	24.0	0.0	131.4	–119.6	142.2	–124.2	142.2	–103.3
Sp 1 – 0.9 pt.	27.0	–112.3	63.2	–154.0	54.8	–195.6	54.0	–229.5
Sp 1 – 1.0 pt.	30.0	–249.4	51.1	–248.2	60.9	–354.6	60.0	–403.3
Sp 2 – 0.0 pt.	30.0	–249.3	51.1	–248.2	61.0	–354.6	60.0	–403.3
Sp 2 – 0.1 pt.	33.0	–137.1	80.1	–176.7	38.9	–233.0	52.7	–244.2
Sp 2 – 0.2 pt.	36.0	–49.8	142.3	–151.3	153.9	–179.3	153.9	–179.3
Sp 2 – 0.3 pt.	39.0	12.5	209.6	–126.1	218.7	–149.4	205.7	–149.4
Sp 2 – 0.4 pt.	42.0	49.9	237.0	–100.9	251.6	–119.5	226.3	–119.5
Sp 2 – 0.5 pt.	45.0	62.4	238.9	–94.6	242.1	–91.4	205.5	–90.0

TABLE 15.37 Total Area Estimation at 7/10th Point of Span 1

Bar Size	L_{dev} (ft.)	$L_{TP\text{-}7/10th\ pt}$(ft.)	$\%_{Effective}$ ($p \times 100$)	No. of Bars per 12-ft. Slab (n)	A_{sb} per Bar	$A_{s\text{-bar}}$ (n p A_{sb})	$A_{s\text{-total}}$ $\sum A_{s\text{-bar}}$	Centroid of rebars $\frac{\sum A_{s\text{-bar}} \times d}{\sum A_{s\text{-bar}}}$
#8 (bottom)	1.53	4.50	100	8	0.79	6.32	24.58	0.605
#11 (bottom)	3.04	2.50	82.23	8	1.56	10.26		
#9 (bottom)	2.72	9.00	100	8	1.00	8.00		
#10 (top)	3.46	23	100	8	1.27	10.16	10.16	0.635

TABLE 15.38 Moment Capacity Estimation at 7/10th Point of Span 1

Demand	Area (A_s)	Depth of Centroid d_s (in.)	Flange Width b (in.)	$a = \frac{A_s \times f_y}{0.85\, f'_c b}$	$\varphi M_n = 0.9 A_s f_y \left(d_s - \frac{a}{2}\right)$
Positive moment	24.58	13.895	144	2.678	925.9
Negative moment	10.16	13.865	144	1.107	406.1

(a) Rating at 7/10th Point of Span 1

At this analysis point, HS20 and permit trucks create both positive and negative moment demands and therefore the rating factor based on positive and negative demands of the trucks need to be established. Whenever the demands of the live load and dead load are acting together, A_1 should be taken as 1.3, and whenever they are acting opposite to each other, A_1 should be taken as 1.0.

First, the area of effective rebar area (after considering the effect of development length) is established and is listed in Table 15.37. Of the three top reinforcement, only #10 bar is fully developed at this point. The #9 top bars are not available at 7/10th point. Of the three bottom reinforcements, #9 bar is fully effective, and #7 bar is not available.

Positive and negative moment capacities are then established and the calculations are listed in a Table 15.38.

Centroid of the top rebar location from bottom $d_s = t_{slab}$ – clear cover – rebar centroid = 16.5 – 2.00 – 0.635 = 13.865 in.

Centroid of the bottom rebar location from top $d_s = t_{slab}$ – clear cover – rebar centroid = 16.5 – 2.00 – 0.605 = 13.895 in.

Now that the capacity, dead load, and live load demands are established, the rating factor can be then established using both positive and negative moment demands using the MBE Equation 6B5.1-1.

Established rating factor at 7/10th point of the span 1 is listed in Table 15.39.

(b) Rating at All 10th Points

To obtain the critical rating of the bridge, rating analysis is performed at all 10th points of the span. Furthermore, since this bridge is symmetrical bridge, the bridge rating is performed at all 10th points within first half of the bridge. Tables 15.40 and 15.41 show the rating based on negative and positive moments, respectively. The most critical rating occurred at 4/10th point of span 1 location and is based on positive moment demand.

TABLE 15.39 Rating Calculations at 7/10th Point of Span 1

Type of Rating	Load Type	Capacity (kip-ft.)	Dead Load Moment (kip-ft.)	Live Load LL + I Demand (kip-ft.)	Rating Factor $RF = \frac{C - A_1 D}{A_2 \times (LL + I)}$	
Inventory rating	Positive moment	925.9	87.2	221.6	1.692	$A_1^a = 1.3$ or 1.0; $A_2 = 2.17$
	Negative moment	–406.1	87.2	–104.6	2.172	
Operating rating	Positive moment	925.9	87.2	221.6	2.821	$A_1^a = 1.3$ or 1.0; $A_2 = 1.3$
	Negative moment	–406.1	87.2	–104.6	3.627	
P5 permit rating	Positive moment	925.9	87.2	240.3	2.601	
	Negative moment	–406.1	87.2	–108.7	3.491	
P13 permit rating	Positive moment	925.9	87.2	240.3	2.601	
	Negative moment	–406.1	87.2	–90.4	4.197	

[a] Whenever the dead load and live load demands are of opposite signs, A_1 is taken as 1.0.

TABLE 15.40 Rating Calculations at all 10th Points Based on Negative Moment Demand

Span, 10th Point	Dead Load (kip-ft.)	A_sTop (in.^{2})	Neg Moment Capacity (kip-ft.)	HS20 + I (kip-ft.)	5-Axle Permit + I (kip-ft.)	13-axle permit + I (kip-ft.)	Inventory RF	Operating RF	P5 RF	P13 RF
Sp 1 – 0.1 pt.	87.3	10.160	406.0	–14.9	–15.5	–12.9	15.25	25.47	24.48	29.42
Sp 1 – 0.2 pt.	149.6	10.160	406.0	–29.9	–31.0	–25.8	8.56	14.29	13.79	16.57
Sp 1 – 0.3 pt.	187.1	10.160	406.0	–44.8	–46.6	–38.7	6.10	10.18	9.79	11.79
Sp 1 – 0.4 pt.	199.5	10.160	406.0	–59.8	–62.1	–51.6	4.66	7.79	7.50	9.03
Sp 1 – 0.5 pt.	187.0	10.160	406.0	–74.7	–77.6	–64.5	3.66	6.11	5.88	7.07
Sp 1 – 0.6 pt.	149.6	10.160	406.0	–89.7	–93.1	–77.4	2.85	4.76	4.59	5.52
Sp 1 – 0.7 pt.	87.2	10.160	406.0	–104.6	–108.7	–90.4	2.17	3.63	3.49	4.20
Sp 1 – 0.8 pt.	0.0	11.334	406.0	–119.6	–124.2	–103.3	1.74	2.90	2.79	3.36
Sp 1 – 0.9 pt.	–112.3	19.334	450.8	–154.0	–195.6	–229.5	1.79	2.99	2.35	2.00
Sp 1 – 1.0 pt.	–249.4	26.160	743.7	–248.2	–354.6	–403.3	1.21	2.02	**1.42**	**1.25**
Sp 2 – 0.0 pt.	–249.3	26.160	977.1	–248.2	–354.6	–403.3	1.21	2.02	1.42	1.25
Sp 2 – 0.1 pt.	–137.1	20.507	977.1	–176.7	–233.0	–244.2	1.58	2.64	2.00	1.91
Sp 2 – 0.2 pt.	–49.8	14.854	784.9	–151.3	–179.3	–179.3	1.58	2.63	2.22	2.22
Sp 2 – 0.3 pt.	12.5	10.160	582.2	–126.1	–149.4	–149.4	1.53	2.55	2.15	2.15
Sp 2 – 0.4 pt.	49.9	10.160	406.0	–100.9	–119.5	–119.5	2.08	3.48	2.93	2.93
Sp 2 – 0.5 pt.	62.4	10.160	406.0	–94.6	–91.4	–90.0	2.28	3.81	3.94	4.00

Note: Whenever the dead load and live load demands are of opposite signs, the dead load factor A_1 is taken as 1.0.

TABLE 15.41 Rating Calculations at All 10th Points Based on Positive Moment Demand

Span, 10th Point	Dead Load (kip-ft.)	A_s Top (in.^{2})	Pos. Moment Capacity (kip-ft.)	HS20 + I (kip-ft.)	5-Axle Permit + I (kip-ft.)	13-Axle Permit + I (kip-ft.)	Inventory RF	Operating RF	P5 RF	P13 RF
Sp 1 – 0.1 pt.	87.3	14.32	566.4	155.9	179.4	179.4	1.78	2.96	2.58	2.58
Sp 1 – 0.2 pt.	149.6	18.425	714.4	253.1	283.8	283.8	1.46	2.45	2.18	2.18
Sp 1 – 0.3 pt.	187.1	26.80	999.1	299.6	335.5	335.5	1.16	1.94	1.73	1.73
Sp 1 – 0.4 pt.	199.5	26.80	999.1	301.6	352.4	352.4	**1.13**	**1.89**	1.61	1.61
Sp 1 – 0.5 pt.	187.0	26.80	999.1	279.5	332.4	332.4	1.25	2.08	1.75	1.75
Sp 1 – 0.6 pt.	149.6	26.80	999.1	271.2	302.7	302.7	1.37	2.28	2.04	2.04
Sp 1 – 0.7 pt.	87.2	26.80	999.1	221.6	240.3	240.3	1.69	2.82	2.60	2.60
Sp 1 – 0.8 pt.	0.0	24.583	925.9	131.4	142.2	142.2	1.96	3.28	3.03	3.03
Sp 1 – 0.9 pt.	–112.3	14.156	560.3	63.2	54.8	54.0	3.18	5.31	6.12	6.22
Sp 1 – 1.0 pt.	–249.4	8.00	324.0	51.1	60.9	60.0	5.17	8.63	7.24	7.35
Sp 2 – 0.0 pt.	–249.3	8.00	324.0	51.1	61.0	60.0	4.02	6.72	5.63	5.72
Sp 2 – 0.1 pt.	–137.1	4.80	196.9	80.1	38.9	52.7	1.92	3.21	6.60	4.88
Sp 2 – 0.2 pt.	–49.8	4.80	196.9	142.3	153.9	153.9	1.60	2.68	2.48	2.48
Sp 2 – 0.3 pt.	12.5	11.12	445.9	209.6	218.7	205.7	1.45	2.43	2.33	2.47
Sp 2 – 0.4 pt.	49.9	17.44	678.0	237.0	251.6	226.3	1.19	1.99	1.87	2.08
Sp 2 – 0.5 pt.	62.4	17.44	678.0	238.9	242.1	205.5	1.15	1.92	1.90	2.23

Note: Whenever the dead load and live load demands are of opposite signs, the dead load factor A_1 is taken as 1.0.

The controlling (lowest) inventory rating factor and operating rating factor for HS20 loading are 1.13 and 1.89, respectively. The operating rating factors for 5-axle permit and 13-axle permit trucks are 1.42 and 1.25, respectively.

B. LRFR Method

To make the analysis consistent between the LFR and LRFR methods, demand and capacity of 12-ft. strip are used to establish the rating factor.

B.1. Dead Load Analysis

Dead load of 12-ft.-wide slab has already been established within LFR method.

B.2. Live Load Analysis

The approximate method of analysis listed in Article 4.6.2.3 of LRFD Specification is used to establish "Equivalent Slab width" (or "E" width) that carries one lane of live loads.

The width of equivalent slab width for single-lane loaded and two-or-more lane loaded cases are

$$E_{\text{one}} = 10 + 5\sqrt{L_1 W_1}\ \text{in.} \le \frac{12W}{N_L} \text{ and } L_1 \le 60 \text{ ft. and } W_1 \le 30 \text{ ft. (LRFD Article 4.6.2.3-1)}$$

$$\text{and } E_{\text{multi}} = 84 + 1.44\sqrt{L_1 W_1}\ \text{ in.} \le \frac{12W}{N_L} \text{ and } L_1 \text{ and } W_1 \le 60 \text{ ft. (LRFD Article 4.6.2.3-2)}$$

For this slab bridge, $N_L = 2$, $L_1 = 30.0$ ft., $W_1 = 34$ ft.

$$E_{\text{one}} = 10 + 5\sqrt{30 \times 30} = 160.0 \text{ in.}$$

$$E_{\text{multi}} = 84 + 1.44\sqrt{30 \times 34} = 130 \le \frac{12 \times 30}{2} = 180.0 \text{ in.}$$

Therefore, E_{one} = 160.0 in. (13.333 ft.) and E_{multi} = 130 in. (10.833 ft.)

The number of HL93 lanes carried by 12-ft. slab = 12/10.833 = 1.108 lanes.

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TABLE 15.42 Dead and Live Load Moment Demands on 12-ft.-Wide Slab Strip

			Maximum Moment Demand with Impact (kip-ft.)					
Location	Distance from Support 1 (ft.)	DL_M	HL93 + I (Pos)	HL93 + I (Neg)	P5 + I (Pos)	P5 + I (Neg)	P13 + I (Pos)	P13 + I (Neg)
Sp 1 – 0.0 pt.	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Sp 1 – 0.1 pt.	3.0	87.3	200.2	−20.5	195.1	−16.8	131.5	−9.5
Sp 1 – 0.2 pt.	6.0	149.6	339.7	−41.0	309.1	−33.5	209.5	−19.2
Sp 1 – 0.3 pt.	9.0	187.1	421.3	−61.5	367.0	−50.3	248.1	−28.7
Sp 1 – 0.4 pt.	12.0	199.5	449.5	−82.0	384.4	−67.0	259.7	−38.3
Sp 1 – 0.5 pt.	15.0	187.0	440.5	−102.4	362.3	−83.8	244.8	−47.9
Sp 1 – 0.6 pt.	18.0	149.6	395.9	−122.9	330.2	−100.5	223.4	−57.5
Sp 1 – 0.7 pt.	21.0	87.2	310.7	−143.3	262.7	−117.3	177.5	−67.0
Sp 1 – 0.8 pt.	24.0	0.0	193.6	−163.9	154.5	−134.0	104.1	−76.6
Sp 1 – 0.9 pt.	27.0	−112.3	87.6	−210.7	59.0	−215.0	39.9	−170.2
Sp 1 – 1.0 pt.	30.0	−249.4	66.1	−346.1	65.5	−387.2	44.3	−299.2
Sp 2 – 0.0 pt.	30.0	−249.3	66.2	−346.0	65.6	−387.2	44.4	−299.2
Sp 2 – 0.1 pt.	33.0	−137.1	94.9	−239.2	40.6	−251.2	37.5	−181.1
Sp 2 – 0.2 pt.	36.0	−49.8	195.4	−198.6	166.2	−196.7	112.4	−133.1
Sp 2 – 0.3 pt.	39.0	12.5	293.6	−170.8	235.1	−163.9	151.6	−110.9
Sp 2 – 0.4 pt.	42.0	49.9	351.8	−143.0	268.9	−131.1	166.4	−88.7
Sp 2 – 0.5 pt.	45.0	62.4	366.4	−115.2	259.1	−98.3	150.8	−66.5

The number of routine P5 truck lanes carried by 12-ft. slab = 12/10.833 = 1.108 lanes.

MBE requires that when evaluating bridges for "special permits," single lane distribution without the MPFs should be used.

So, the number of special P13 truck lanes carried by 12-ft. slab = 12/(13.333 × 1.2) = 0.750 lanes.

The live load demands are obtained using two-dimensional analysis software. Since the bridge is symmetrical about midspan of the span, results are listed at every 10th point up to midpoint of the span in Table 15.42.

B.3. Moment Capacity Calculation

The values for dead and live loads have already been established. Next step in establishing the rating factor is the capacity of the member. Here, capacity is established at every 10th point of the span.

(a) Effect of Development Length

As stated in the preceding sections A.3.1 and A.3.4 in section 15.3.4, considering the effect of development length of each rebar, the effective bar area at each analysis point is established. Table 15.43 shows the required development length of bars and effective areas are listed in Table 15.44.

(b) Moment Reduction Factor

Within LRFD specification, the moment reduction factor varies with the tensile strain in the tensile rebar. The strain at the centroid of the tension bars can be established by the following equation (Article C5.7.2.1-1 and Equation 5.7.3.1.2-4):

$$\varepsilon_t = \frac{0.003(d_s - c)}{c}, \text{where } c = \frac{A_s f_y}{0.85\beta_1 f'_c b}$$

For strain $\varepsilon_t \geq 0.005$, moment reduction factor φ is 0.90; for $0.005 \geq \varepsilon_t \geq 0.002$, moment reduction factor

φ is $0.65 + 0.15\left(\dfrac{d_t}{c} - 1\right)$; and for strain $\varepsilon_t \leq 0.002$, moment reduction factor φ is 0.75.

Centroid of the rebar location from bottom = t_{slab} – clear cover – $1/2d_b$ = 16.5 – 2 – 1.25/2 = 13.875 in.

Centroid of the rebar location from top = t_{slab} – clear cover – $1/2d_b$ = 16.5 – 2 – 1.25/2 = 13.875 in.

TABLE 15.43 Required Development Length for LRFR Method

Bar (Location)	Area (in.^{2})	Diameter d_b (in.)	Basic Development (in.)	Factor for Depth	Factor for Spacing ≥ 6 in.	Required Development L_{dev}
#9 (top bar)	1.128	1.00	28.92	1.4	0.8	32.39 in. = 2.70 ft.
#10 (top bar)	1.270	1.27	36.72	1.4	0.8	41.13 in. = 3.43 ft.
#8 (bottom)	1.000	0.79	22.80	1.0	0.8	18.24 in. = 1.52 ft.
#11 (bottom bar)	1.410	1.56	45.00	1.0	0.8	36.00 in. = 3.00 ft.
#7 (bottom)	0.875	0.60	17.28	1.0	0.8	13.82 in. = 1.15 ft.

TABLE 15.44 Moment Capacity of 12-ft. Slab Strip at Every 10th Point of the Spans

Span, 10th Point	A_{st}Top (in.^{2})	c (in.)	ε_t	φ	φM_n (kip-ft.)	A_{sb}Bottom (in.^{2})	c (in.)	ε_t	φ	φM_n (kip-ft.)
Sp 1 – 0.1 pt.	10.160	1.302	0.029	0.90	406.0	14.32	1.835	0.020	0.90	562.6
Sp 1 – 0.2 pt.	10.160	1.302	0.029	0.90	406.0	18.48	2.368	0.015	0.90	713.4
Sp 1 – 0.3 pt.	10.160	1.302	0.029	0.90	406.0	26.80	3.435	0.009	0.90	998.2
Sp 1 – 0.4 pt.	10.160	1.302	0.029	0.90	406.0	26.80	3.435	0.009	0.90	998.2
Sp 1 – 0.5 pt.	10.160	1.302	0.029	0.90	406.0	26.80	3.435	0.009	0.90	998.2
Sp 1 – 0.6 pt.	10.160	1.302	0.029	0.90	406.0	26.80	3.435	0.009	0.90	998.2
Sp 1 – 0.7 pt.	10.160	1.302	0.029	0.90	406.0	26.80	3.435	0.009	0.90	998.2
Sp 1 – 0.8 pt.	11.334	1.453	0.026	0.90	450.8	24.72	3.168	0.010	0.90	929.1
Sp 1 – 0.9 pt.	19.334	2.478	0.014	0.90	743.7	14.24	1.825	0.020	0.90	559.5
Sp 1.10 pt.	26.160	3.353	0.009	0.90	977.1	8.00	1.025	0.038	0.90	322.5
Sp 2.00 pt.	26.160	3.353	0.009	0.90	977.1	8.00	1.025	0.038	0.90	322.5
Sp 2 – 0.1 pt.	20.507	2.628	0.013	0.90	784.9	4.80	0.615	0.065	0.90	196.0
Sp 2 – 0.2 pt.	14.854	1.904	0.019	0.90	582.2	4.80	0.615	0.065	0.90	196.0
Sp 2 – 0.3 pt.	10.160	1.302	0.029	0.90	406.0	11.12	1.425	0.026	0.90	442.7
Sp 2 – 0.4 pt.	10.160	1.302	0.029	0.90	406.0	17.44	2.235	0.016	0.90	676.2
Sp 2 – 0.5 pt.	10.160	1.302	0.029	0.90	406.0	17.44	2.235	0.016	0.90	676.2

Moment capacity is first established at every 10th point of the spans. Calculations are performed and listed in Table 15.44. A_s is the estimated effective rebar area considering the development length.

B.4. Rating Calculations

$$\text{RF} = \frac{C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_P P}{\gamma_L\, L(1+I)} \tag{15.15}$$

The load factors for 5-axle routine permit trucks and special 13-axle permit trucks are 1.58 and 1.50, respectively. The derivation of the load factors is explained in Section 15.3.1.

The load factor γ_{DC} is 1.25 and γ_{DW} is 1.5. In the preceding equations, minimum load factor for dead load is used whenever the dead and live load demands have opposite signs.

By substituting these values and appropriate load effect values, the positive moment and negative moment rating could be estimated. The calculations and results are given in Table 15.45.

The inventory rating and operating rating based on HL93 are 0.93 and 1.21, respectively. Permit rating for 5-axle routine permit truck and for 13-axle special permit truck are 1.09 and 1.48, respectively.

15.3.6 Two-Span Continuous Prestressed Precast Concrete Box Beam Bridge

Given: Typical section and elevation of three continuous span precast prestressed box beam bridge is shown in Figure 15.12a. The span length of each span is 120 ft. (36.5 m), 133 ft. (40.6 m), and 121 ft. (36.9 m). Total width of the bridge is 82 ft. (25 m) and number of precast prestressed box girders are placed at a spacing of 10 ft. (3.1 m). Cross section of the box beam and tendon profile of the girder is

TABLE 15.45 LRFR Factors of 12-ft. Slab Strip at Every 10th Point of the Span

Span, 10th Point	φM_n (kip-ft.)	φM_n (kip-ft.)	DL_M (kip-ft.)	HL93 (kip-ft.)		HL93 RF		P5 Demand (kip-ft.)		P5 RF	P13 Demand (kip-ft.)		P13 RF
	Negative	Positive		Positive	Negative	Inventory RF	Operating RF	Positive	Negative		Positive	Negative	
Sp 1 – 0.1 pt.	406.0	562.6	87.3	200.2	–20.5	1.72	2.24	195.1	–16.8	1.96	131.5	–9.5	3.06
Sp 1 – 0.2 pt.	406.0	713.4	149.6	339.7	–41.0	1.36	1.77	309.1	–33.5	1.66	209.5	–19.2	2.58
Sp 1 – 0.3 pt.	406.0	998.2	187.1	421.3	–61.5	1.04	1.34	367.0	–50.3	1.32	248.1	–28.7	2.05
Sp 1 – 0.4 pt.	406.0	998.2	199.5	449.5	–82.0	0.95	1.23	384.4	–67.0	1.23	259.7	–38.3	1.92
Sp 1 – 0.5 pt.	406.0	998.2	187.0	440.5	–102.4	0.99	1.29	362.3	–83.8	1.34	244.8	–47.9	2.08
Sp 1 – 0.6 pt.	406.0	998.2	149.6	395.9	–122.9	1.17	1.52	330.2	–100.5	1.55	223.4	–57.5	2.42
Sp 1 – 0.7 pt.	406.0	998.2	87.2	310.7	–143.3	1.51	1.96	262.7	–117.3	1.98	177.5	–67.0	3.08
Sp 1 – 0.8 pt.	450.8	929.1	0.0	193.6	–163.9	1.57	2.04	154.5	–134.0	2.13	104.1	–76.6	3.58
Sp 1 – 0.9 pt.	743.7	559.5	–112.3	87.6	–210.7	1.64	2.12	59.0	–215.0	1.78	39.9	–170.2	2.36
Sp 1 – 10 pt.	977.1	322.5	–249.4	66.1	–346.1	1.10	1.42	65.5	–387.2	**1.09**	44.3	–299.2	**1.48**
Sp 2 – 0.0 pt.	977.1	322.5	–249.3	66.2	–346.0	1.10	1.42	65.6	–387.2	**1.09**	44.4	–299.2	**1.48**
Sp 2 – 0.1 pt.	784.9	196.0	–137.1	94.9	–239.2	1.47	1.90	40.6	–251.2	1.55	37.5	–181.1	2.26
Sp 2 – 0.2 pt.	582.2	196.0	–49.8	195.4	–198.6	1.40	1.82	166.2	–196.7	1.67	112.4	–133.1	2.60
Sp 2 – 0.3 pt.	406.0	442.7	12.5	293.6	–170.8	1.29	1.67	235.1	–163.9	1.60	151.6	–110.9	2.50
Sp 2 – 0.4 pt.	406.0	676.2	49.9	351.8	–143.0	1.00	1.29	268.9	–131.1	1.44	166.4	–88.7	2.46
Sp 2 – 0.5 pt.	406.0	676.2	62.4	366.4	–115.2	**0.93**	**1.21**	259.1	–98.3	1.71	150.8	–66.5	2.94

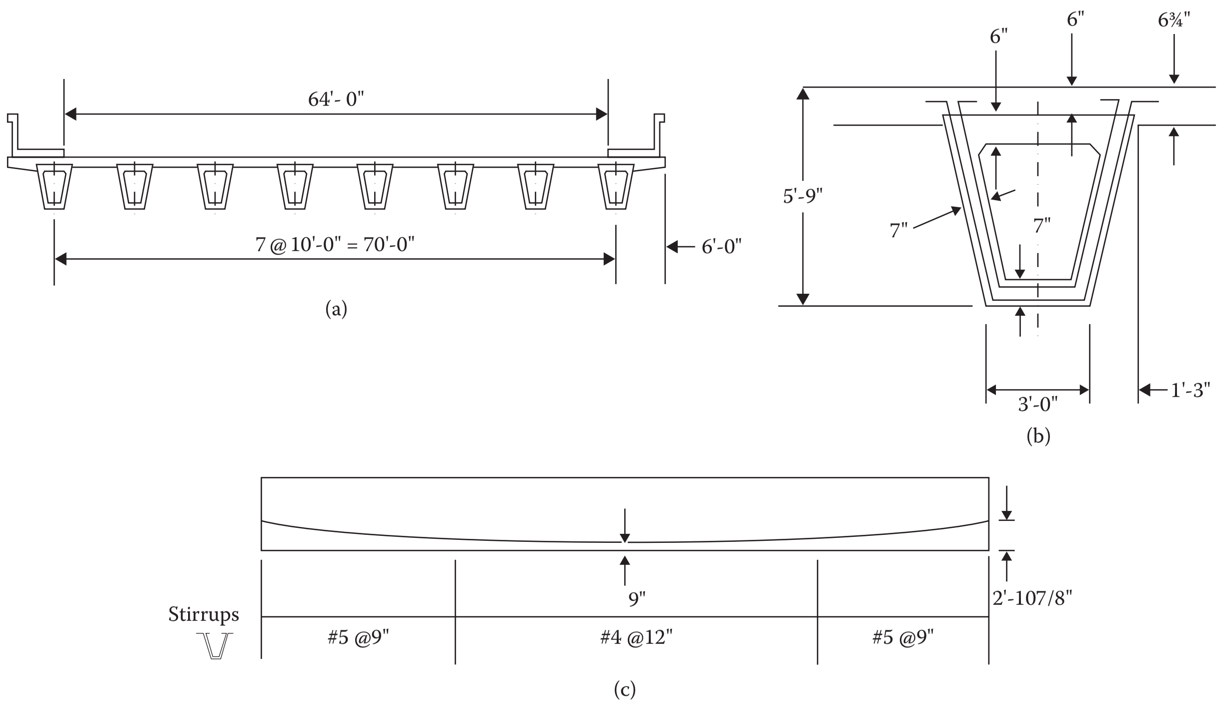


FIGURE 15.12 Details of two-span continuous prestressed precast box beam bridge example: (a) Typical section, (b) beam section details, and (c) prestressing tendon profile.

shown in Figure 15.12b, c. Each barrier rails weighs 1268 lb/ft. (18.5 N/mm). Information gathered from the plans is: (1) f_c' of the girder and slab is 5500 psi and 3500 psi, respectively. (2) Working force (total force remaining after losses including creep) = 2020 kip. (3) x at midspan = 9 in. Assume that (1) the bridge was made continuous for live loading. (2) No temporary supports were used during the erection of the precast box beams. (3) Properties of the precast box are: area = 1,375 in.^{2}; moment of inertia = 30.84 ft.^{4}; Y_t = 28.58 in.; Y_b = 34.4 in. (4) F_y of reinforcing steel is 40,000 psi.

Required: (a) Rate the interior girder of span 2 for HS20 vehicle using LFR method.

Solution:

1. Dead Load Calculations

Self-weight of the box beam = (1375/144)(0.15) = 1.43 kip/ft.

Weight of slab (tributary area approach) = (6.75/12)(10)(0.15) = 0.85 kip/ft.

Total dead weight on the box beam = 2.28 kip/ft. (33.2 N/mm).

Contribution of barrier rail on box beam = 2(1.268/8) = 0.318 kip/ft.

Thus, total additional dead load on the box beam = 0.318 kip/ft. (4.6 N/mm).

Girder is simply supported for dead loads, thus, maximum dead load moment.

= (2.28) (133^2/8) = 4926 kip-ft. (6.68 MN-m).

2. Live Load Calculations

According to Article 3.28 of Standard Specification, distribution factor (DF) for interior spread box beam is given by

$$\mathrm{DF} = \left(\frac{2\,N_L}{N_B}\right) + k\left(\frac{S}{L}\right)$$

Where N_L = number of traffic lanes = 64/12 = 5 (no fractions); N_B = number of beams = 8

S = girder spacing = 8 ft.; L = span length = 133 ft.; W = roadway width = 64 ft.

$k = 0.07\ W – N_L\ (0.10\ N_L – 0.26) – 0.2\ N_B – 0.12 = 1.56$

Thus, $\mathrm{DF} = \left(\frac{2 \times 5}{8}\right) + 1.56\left(\frac{10}{133}\right) = 1.37$ wheels

3. Demands on the Girder

Load demands are estimated using a two-dimensional analysis and summary is given in Table 15.46.

4. Section Property Calculations

To estimate the stresses on the prestressed box beam, the section properties for composite girder need to be estimated. Calculations of the composite girder properties are done separately and the final results are listed in Table 15.47.

5. Stress Calculations

Stresses at different fiber locations are calculated using $\left(\frac{P}{A}\right) + \left(\frac{M\,c}{I}\right)$ expression. The summary of the results at midspan and at bent 2 locations is given in Tables 15.48 and 15.49, respectively.

TABLE 15.46 Load Demands for Prestressed Precast Box Beam Bridge Example

Description	At Midspan	At Bent 2	At Bent 3
Dead load moment (kip-ft.)	4224	0	0
Additional dead load moment (kip-ft.)	194	–506	–513
HS20 moment with impact (kip-ft.)	1142	–1313	–1322
Dead load shear (kip)	0.0	153.6	–153.6
Additional dead load shear (kip)	0.0	21.1	–21.2
HS20 positive shear (moment)[a](kip)	24.8 (1104)	61.1 (–974)	7.1 (127)
HS20 negative shear (moment)[a](kip)[a]	–24.8 (1104)	–7.1 (131)	–61.2 (–980)

[a] Values within the bracket indicate the moment corresponds to the reported shear.

TABLE 15.47 Section Properties for Prestressed Precast Box Beam Bridge Example

Description	Area (in.^{2})	Moment of Inertia (ft.^{4})	Y Bottom of Girder (in.)	Y Top of Girder (in.)	Y Top of Slab (in.)
For dead loads	1375	30.84	34.42	28.58	NA
For additional dead loads	1578	39.22	38.55	24.45	30.45
For live loads	1984	50.75	44.23	18.77	24.77

TABLE 15.48 Stresses at Midspan for Prestressed Precast Box Beam Bridge Example

Location = Midspan	Stresses in the Box Beam (psi)			
Load description	At Top Concrete Fiber	At Bottom Concrete Fiber	At Centroid of Composite Box Beam Concrete Fiber	At Prestress Tendon
Dead load (self + slab)	2265	–2728	777	20.15
Prestress P_{eff} = 2020 kip; (e = 25.42 in.)	–1615	3443	–108	147.1
Additional dead (barrier)	70	–110	16	0.845
Live load	244	–575	0	4.59
Live load moment for shear	236	–556	0	4.43

TABLE 15.49 Stresses at Bent 2 for Prestressed Precast Box Beam Bridge Example

Location = Bent 2	Stresses in the Box Beam (psi)				
Load Description	At Top Concrete Fiber	At Bottom Concrete Fiber	At Centroid of Composite Box Beam Concrete Fiber	At Top of Slab Fiber	At Prestress Tendon
Dead load (self + slab)	0	0	0	0	0
Prestress P_{eff} = 2020 kip; (e = 12 in.)	680	680	680	0	167.5
Additional dead (barrier)	–183	288	–4	–228	–0.3
Live load	–281	662	0	–371	–1.47
Live load moment for positive shear	–208	491	0	–274	–1.08

6. Capacity Calculations

6.1. Moment Capacity at Midspan

The actual area of steel could only be obtained from the shop plans. Since the shop plans are not readily available, the following approach is used.

Assume the total loss including the creep loss = 35 ksi (241.3 MPa).

$$\text{Thus, the area of prestressing steel} = \frac{\text{Working force}}{0.75\times 270-35} = \frac{2020}{167.5} = 12.06\ \text{in.}^2\left(7781\ \text{mm}^2\right)$$

b_{eff} = 120 in.; t_s = 6.75 in.; d_p = (5.75)(12) – 9 in. = 60 in.; b_w = 14 in.

$$\rho^* = \frac{A_s^*}{bd} = \frac{12.06}{120\times 60} = 0.001675$$

$$f_{su}^* = f_s\left(1-\frac{0.5\rho^*\ f_s'}{f_c'}\right) = 270\left(1-\frac{0.5\times 0.001675\times 270}{5.5}\right) = 258.9\ \text{ksi (1785 MPa)}$$

Neutral axis location $= 1.4\,d\rho^* \dfrac{f_{su}^*}{f_c'} = 1.4\times 60\times 0.001675\times \dfrac{258.9}{5.5} = 6.62$ in. $< t_s = 6.75$ in. Since the neutral axis falls within the slab, this girder can be treated as a rectangular section for moment capacity calculations.

$$R = \varphi\ M_n = \varphi\, A_s^*\, f_{su}^* d\left(1 - 0.6\,\rho\frac{f_{su}^*}{f_c'}\right) \text{ and } \varphi = 1.00 = 14873.1 \text{ kip-ft. (20.17 MN-m)}$$

6.2. Moment Capacity at the Face of the Support

15 #11 bars are used at top of the bent, thus total area of steel = (15)(1.56) = 23.4 in.^{2}

Depth of the reinforcing steel from the top of compression fiber = 69 – 1.5 – 1.41/2 = 66.795 in. (1696.6 mm).

$F_y = 40$ ksi, resistance reduction factor $\varphi = 0.90$.

Then, the moment capacity $\varphi M_n = 4474.2$ kip-ft. (8.88 MN-m) (*based on rectangular section*).

6.3. Shear Capacity at Midspan

Standard Specification Section 9.20 addresses the shear capacity of a section. Shear capacity depends on the cracking moment of the section. When the live load causes tension at bottom fiber, cracking moment is to be calculated based on the bottom fiber stress. On the contrary, when the live load causes tension at top fiber of the beam, cracking moment is to be calculated based on the top fiber stress.

At midspan location, the moment reported with the maximum live load shear is positive. Positive moments will induce tension at the bottom fiber and thus cracking moment is to be based on the stress at bottom fiber.

$f'_c = 5500$ psi and from Table 15.48, f_{pe} at midspan bottom fiber = 3443 psi; f_d at bottom fiber = –2728 – 110 = –2838 psi; f_{pc} at centroid = 777 – 108 + 16 = 685 psi

$$M_{cr} = \frac{I}{Y_t}\left(6\sqrt{f_c'} + f_{pe} - f_d\right) = \frac{50.75\times 12^4}{44.23}\left(6\sqrt{5500} + 3443 - 2838\right)\left(\frac{1}{12{,}000}\right) = 2081 \text{ kip-ft.}$$

Factored total moment $M_{max} = 1.3\ M_D + (1.3)(1.67)\ M_{LL+I} = 1.3\ (4224 + 194) + 2.167\ (1104) =$ 8136 kip-ft.

Factored total shear $V_i = 1.3\ (0 + 0) + 2.167\ (24.8) = 53.7$ kip

$V_d = 0$ kip; $b_w = 14$ in.; $d = 60$ in.; $f_{pc} = 685$ psi

$$V_{ci} = 0.6\sqrt{f_c'}\,b_w d + V_d + \frac{V_i M_{cr}}{M_{max}} = 0.6\sqrt{5500}\times 14\times 60\times\left(\frac{1}{1000}\right) + 0 + \frac{53.7\times 2081}{8136} = 51.2 \text{ kip } (227.7 \text{ kN})$$

(controls—since lesser than V_{cw})

$$V_{cw} = \left(3.5\sqrt{f_c'} + 0.3 f_{pc}\right)b_w d + V_p = \left(3.5\sqrt{5500} + 0.3\times 685\right)14\times 60\times\left(\frac{1}{1000}\right) + 0 = 390 \text{ kip } (1734 \text{ kN})$$

$V_c = 51.2$ kip (227.7 kN) (lesser than V_{ci} and V_{cw})

$$V_s = 2A_v\frac{F_y d_s}{S} = 4\times 0.20\times\frac{40\times 60}{12} = 160 \text{ kip } (711.7 \text{ kN})$$

Shear capacity at midspan $V_u = \varphi\,(V_c + V_s) = 0.85\,(51.2 + 160) = 179$ kip (796.1 kN)

6.4. Shear Capacity at the Face of Support at Bent 2

Negative shear reported at this location is so small and thus rating will not be controlled by the negative shear at bent 2. Moment reported with the positive shear is negative and thus the following calculations are based on the stress at top fiber.

From Table 15.49, f_d at top of slab fiber = –228 psi.

And, f_{pe} at support top of slab fiber (slab poured after prestressing) = 0 psi.

$$M_{cr} = \frac{I}{Y_t}\left(6\sqrt{f_c'} + f_{pe} - f_d\right) = \frac{50.75\times 12^4}{44.23}\left(6\sqrt{3500} + 0 - 228\right) = 252 \text{ kip-ft.}$$

$V_d = 153.6 + 21.1 = 174.7$ kip; $b_w = 14$ in.; $d = 69 - 1.5 - 1.41/2 = 66.795$ in.; $f_{pc} = 676$ psi

Factored total moment $M_{max} = 1.3\,M_D + (1.3)(1.67)\,M_{LL+I}$

$= 1.3(0 \pm 506) + 2.167(-974) = -2769$ kip-ft.

Factored total shear $V_i = 1.3 \times (153.6 + 21.1) + 2.167 \times (61.1) = 360$ kip

$$V_{ci} = 0.6\sqrt{5500}\times 14\times 66.795\left(\frac{1}{1000}\right) + 0 + \frac{360\times 251.7}{2769} = 74.3 \text{ kip}$$

$$V_{cw} = \left(3.5\sqrt{f_c'} + 0.3 f_{pc}\right) b_w d + V_p = \left(3.5\sqrt{5500} + 0.3\times 676\right)\times 14\times 66.795\left(\frac{1}{1000}\right) + 0 = 432 \text{ kip}$$

$V_c = 74.3$ kip (330.4 kN) (lesser than V_{cw} and V_{ci})

$$V_s = A_v\frac{F_y d_s}{S} = 4\times 0.31\times\frac{40\times 66.695}{6} = 367.6 \text{ kip (1635 kN)}$$

Shear capacity at bent 2, $V_u = \varphi\,(V_c + V_s) = 0.85\,(74.3 + 367.6) = 375.6$ kip (1671 kN)

7. Rating Calculations

The rating calculations for load factor method need to be done using strength and serviceability limit states. Serviceability level rating need not be done at operating level.

7.1. Rating Calculations Based on Serviceability Limit State

Serviceability conditions are listed in AASHTO Design Specification Sections 9.15.1 and 9.15.2.2. These conditions are duplicated in the MBE.

(i) Using the compressive stress under all load combination

The general expression will be $\mathrm{RF}_{\text{INV-COMALL}} = \dfrac{0.6 f_c' - f_d - f_p + f_s}{f_l}$

At midspan, $\mathrm{RF}_{\text{INV-COMALL}} = \dfrac{0.6\times 5500 - (2265 + 70) - (-1615) + 0}{244} = 10.57$

At bent 2 support, $\mathrm{RF}_{\text{INV-COMALL}} = \dfrac{0.6\times 5500 - (0 + 288) - 680 + 0}{662} = 3.52$

(ii) Using the compressive stress of live load, half the prestressing and permanent dead load

The general expression will be $\mathrm{RF}_{\text{INV-COMLIVE}} = \dfrac{0.4 f_c' - f_d - 0.5 f_p + 0.5 f_s}{f_l}$

At midspan, $\mathrm{RF}_{\text{INV-COMLIVE}} = \dfrac{0.4\times 5500 - (2265 + 70) - 0.5(-1615) + 0.5(0)}{244} = 2.76$

At bent 2 support, $\mathrm{RF}_{\text{INV-COMLIVE}} = \dfrac{0.4\times 5500 - (0 + 288) - 0.5(680) + 0.5(0)}{662} = 2.37$

TABLE 15.50 Rating Calculations Prestressed Precast Box Beam Bridge Example

Location	Description	Inventory Rating	Operating Rating
Midspan	Maximum moment	$\frac{14873.1-1.3\times(4224+194)}{1.3\times1.67\times1142}=3.69$	$\frac{14873.1-1.3\times(4224+194)}{1.3\times1142}=6.16$
	Maximum shear	$\frac{179-1.3\times(0+0)}{1.3\times1.67\times24.8}=3.33$	$\frac{179-1.3\times(0+0)}{1.3\times24.8}=5.56$
Bent 2	Maximum moment	$\frac{4474-1.3\times(0+506)}{1.3\times1.67\times1313}=1.34$	$\frac{4474.0-1.3\times(0+506)}{1.3\times1313}=2.24$
	Maximum shear	$\frac{375.6-1.3\times(153.6+21.1)}{1.3\times1.67\times61.1}=1.12$	$\frac{375.6-1.3\times(153.6+21.1)}{1.3\times61.1}=1.87$

(iii) Using the allowable tension in concrete

The general expression will be $RF_{\text{INV-CONTEN}} = \dfrac{6\sqrt{f_c'} - f_d - f_p - f_s}{f_l}$

$$\text{At midspan, } RF_{\text{INV-CONTEN}} = \frac{6\sqrt{5500} - (2728+110) - (-3443) - 0}{575} = 1.826$$

$$\text{At bent 2 support, } RF_{\text{INV-CONTEN}} = \frac{6\sqrt{5500} - (0+183) - (-680) - 0}{281} = 3.352$$

(iv) Using the allowable prestressing steel tension at service level

The general expression will be $RF_{\text{INV-PRETEN}} = \dfrac{0.8 f_y' - f_d - f_p - f_s}{f_l}$

$$\text{At midspan, } RF_{\text{INV-PRETEN}} = \frac{0.8\times270 - 20.99 - (147.1) - 0}{4.59} = 10.43$$

$$\text{At bent 2 support, } RF_{\text{INV-PRETEN}} = \frac{0.8\times270 - (-3.08) - 167.5 - 0}{1.468} = 30.94$$

7.2. Rating Calculations Based on Strength Limit State

The general expression for $RF = \dfrac{\varphi R_n - \gamma_D D}{\gamma_L \beta_L L(1+I)}$

According to AASHTO MBE, γ_D is 1.3, γ_L is 1.3, and β_L is 1.67 and 1.0 for inventory and operating factors, respectively. Rating calculations are made and given in Table 15.50.

8. Summary

The critical inventory rating of the interior girder is controlled by the shear demand at bent support. The critical operating rating of the girder is controlled by moment at bent 2 location.

15.3.7 Simply Supported Precast Prestressed Concrete I-Girder Bridge

Given: A bridge, which was built in 1969, consists of five (5) single simple span precast pretensioned I-girders composite with reinforced concrete deck, on reinforced concrete seat type abutments. The span length is 102 ft. 6½ in. (31.25 m). Typical cross section, elevation, and girder details are shown in Figure 15.13a through d. General notes given in the plan indicated that $f_c' = 3{,}000$ psi (20.7 MPa) for cast-in-place concrete deck; $f_{ci}' = 4{,}000$ psi (27.6 MPa), $f_c' = 5{,}000$ psi (34.5 MPa) for precast I-girders; $f_s = 20{,}000$ psi (137.9 MPa). Assume the weight of steel railing as 40 lb/ft. (0.6 N/mm).

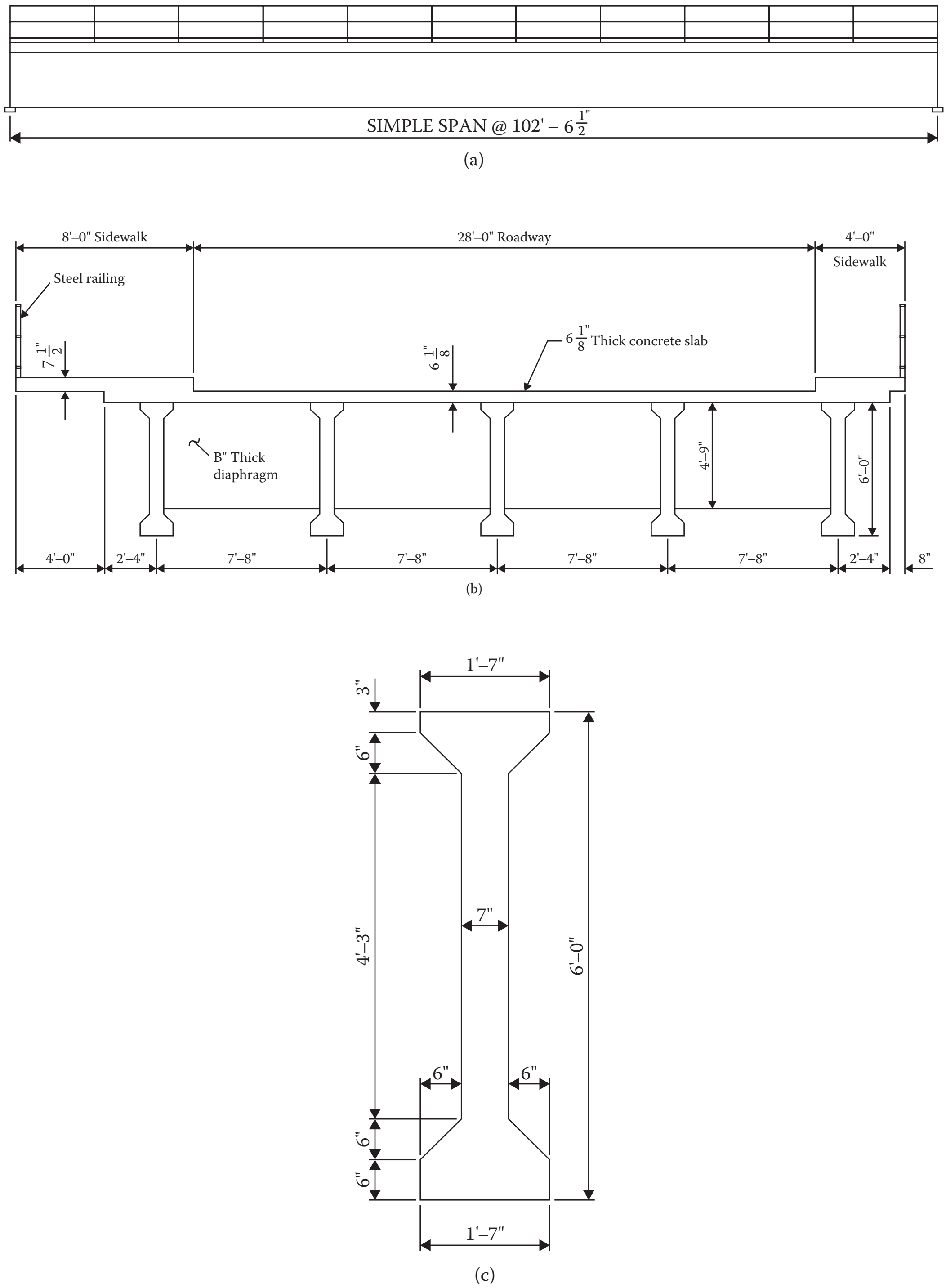
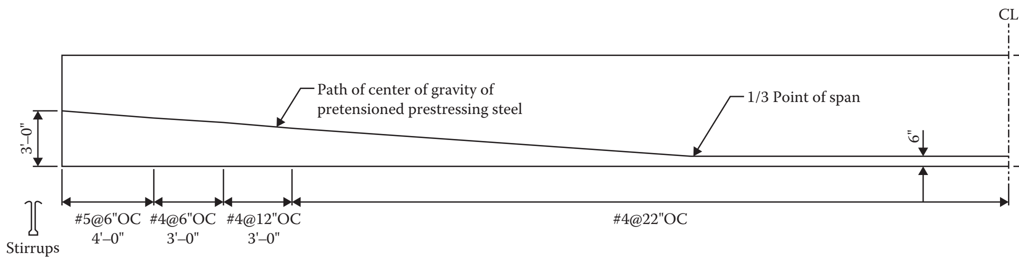
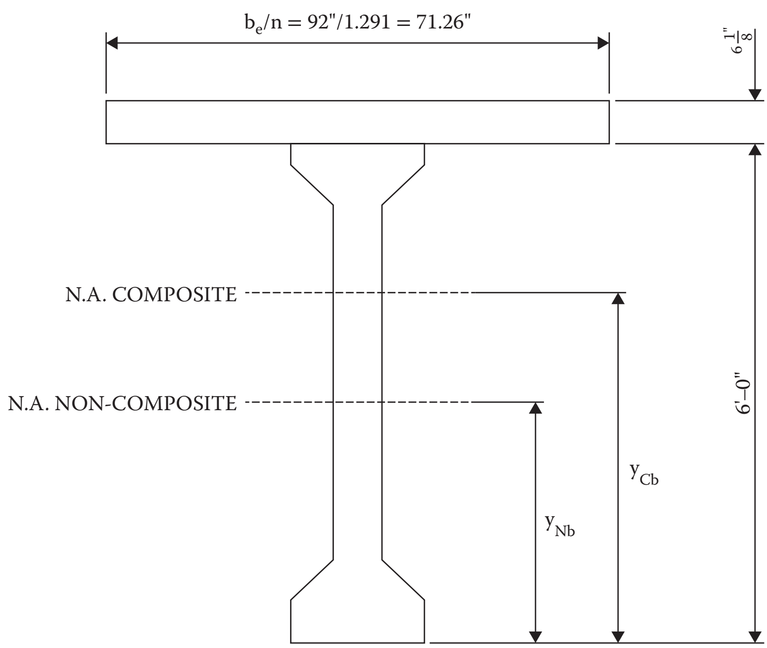


FIGURE 15.13 Details of simply-supported precast prestressed concrete I-girder bridge example. (a) Elevation, (b) typical section, (c) precast girder section.



(d)



(e)

FIGURE 15.13 *(Continued)* Details of simply-supported precast prestressed concrete I-girder bridge example: (d) precast girder prestressing tendon profile and stirrups details and (e) composite section.

Required: Assuming no deterioration of material occurred, determine critical factor for the interior girder of the bridge for the live load vehicle listed in Section 15.3.1 using LRFR method for the following limit states.

1. Service level stress at midspan
2. Ultimate moment at midspan
3. Ultimate shear at a distance of *d* (62½ in.) from the support [end support]
4. Ultimate shear at 1/3 span point

Solution:

1. Section Properties

To determine the section properties of composite section of the concrete deck and prestressed (PS) I-girder, the ratio between modulus of elasticity of concrete of concrete deck and PS I-girder is required.

For concrete strength of girder, $f'_c = 5000$ psi

TABLE 15.51 Section Properties

Noncomposite Section	Composite Section
$y_{Nt} = 37.5$ in.	$y_{Ct} = 27.824$ in.
$y_{Nb} = 34.5$ in.	$y_{Cb} = 50.301$ in.
$I_N = 400{,}653$ in.^{4}	$I_C = 840{,}416$ in.^{4}
$A_N = 684.0$ in.^{2}	$A_C = 1{,}120.5$ in.^{2}

$$E_c = 33 \times 145^{1.5} \times \frac{\sqrt{5000}}{1000} = 4074 \text{ ksi}$$

For concrete strength of deck, $f'_c = 3000$ psi $E_c = 33 \times 145^{1.5} \times \dfrac{\sqrt{3000}}{1000} = 3156$

Therefore, $n = E_c(\text{girder})/E_c(\text{deck}) = 4074/3156 = 1.291$.

The typical composite cross section is shown in Figure 15.13e and the corresponding section properties are shown in Table 15.51.

2. Dead Load Calculations

Self-weight of the PS I-girder = (684/144)(0.15) = 0.712 kip/ft.

Weight of slab (tributary area approach) = (6.125/12)(92/12)(0.15) = 0.587 kip/ft.

Contribution of concrete sidewalk = (7.5/12)(8 + 4)(0.15)/5 = 0.225 kip/ft.

Total dead weight on the PS I-girder = 1.524 kip/ft.

Contribution of barrier rail on the PS I-girder = (0.04 + 0.04)/5 = 0.016 kip/ft.

Thus, total additional dead load on the PS I-girder = 1.54 kip/ft. (22.5 N/mm).

Point load due to weight of diaphragm applied at 1/3th and 2/3th points of the span = (8/12) [(57/12)(92/12)-(471/144)] (0.15) = 3.315 kip (14.7 kN).

Dead load moment at midspan = $\omega L^2/8 + P(L/3) = (1.54)(102.54)^2/8 + (3.315)(102.54)/3 = 2137.4$ kip-ft. (2.9 MN-m).

Dead load shear at 1/3th point of the span = $\omega L/6 + P = (1.54)(102.54)/6 + 3.315 = 29.63$ kip (131.8 kN).

3. Live Load Calculations

The traffic lane width of this bridge is 28.0 ft. According to MBE, any bridge with a minimum traffic lane width of 18 ft. needs to carry two lanes. Furthermore, the demand of the special permit trucks (in this case, 13-axle P13 truck) needs to be established using single-lane LLDF without the MPFs.

Both single-lane loaded LLDF (g_1) and two or more design lanes loaded (g_2) are first established for moment and shear.

LLDF equations for moment demand of precast concrete I sections (type k cross-section) listed in Table 4.6.2.2.2b-1 of the LRFD is as follows:

One-lane loaded distribution factor $g_{1M} = 0.060 + \left(\dfrac{S}{14}\right)^{0.4}\left(\dfrac{S}{L}\right)^{0.3}\left(\dfrac{K_g}{12Lt_s^3}\right)^{0.1}$

Two more lanes loaded distribution factor $g_{2M} = 0.075 + \left(\dfrac{S}{9.5}\right)^{0.6}\left(\dfrac{S}{L}\right)^{0.2}\left(\dfrac{K_g}{12Lt_s^3}\right)^{0.1}$

where $e_g = 72 + 6.125 - 34.5 \text{ in.} - \dfrac{6.125 \text{ in.}}{2} = 40.56 \text{ in.}^2$

$$K_g = n \cdot (I + A \times e_g^2) = 1.291 \times (400{,}653 + 684 \times 40.56^2) = 1{,}969{,}951 \text{ in.}^4$$

So, $$g_{1M} = 0.060 + \left(\frac{7.67}{14}\right)^{0.4}\left(\frac{7.67}{102.54}\right)^{0.3}\left(\frac{1{,}969{,}951}{12\times102.54\times6.125^3}\right)^{0.1} = 0.498 \text{ lane.}$$

And, $$g_{2M} = 0.075 + \left(\frac{7.67}{9.5}\right)^{0.6}\left(\frac{7.67}{102.54}\right)^{0.2}\left(\frac{1{,}969{,}951}{12\times102.54\times6.125^3}\right)^{0.1} = 0.711 \text{ lane.}$$

Therefore, the moment LLDF for the interior girder is greater than g_{1M} and $g_{2M} = 0.711$ lane.

LLDF equations for shear demand of precast concrete I sections listed in Table 4.6.2.2.3a-1 of the LRFD is as follows:

One-lane loaded distribution factor $g_{1V} = 0.36 + \frac{S}{25} = 0.36 + \frac{7.67}{25} = 0.667$ lane.

Two more lanes loaded distribution factor $g_{2V} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 = 0.791$ lane.

Therefore, the shear LLDF for the interior girder is greater than g_{1V} and $g_{2V} = 0.791$ lane.

Furthermore, MBE recommends that the MPF of 1.2 embedded in the LLDF can be removed when estimating the live load demands for "Special" Permit crossing.

The moment LLDF for the interior girder for P13 special permit truck = $\frac{g_{1M}}{1.2} = 0.415$ lane.

The shear LLDF for the interior girder for P13 special permit truck = $\frac{g_{1V}}{1.2} = 0.556$ lane.

Then, the live load demands at midspan, "*d*" distance from support, and 1/3th point of the span can be determined and the results are summarized in Table 15.52.

4. LRFD Service Level Stresses and the Corresponding Ratings at Service

Service III load rating will be performed for HL93 truck inventory level, while service I load rating will be performed for all permit truck cases in this example.

Allowable tensile stress at service (inventory) = $6\sqrt{f_c'} = 6\times\sqrt{5000}$ psi = 424 psi.

Effective final prestressing force per strand, $P_{se} = 720$ kip.

Eccentricity of prestress force at midspan, $e = 34.5 - 6$ in. = 28.5 in.

The midspan dead load moment due to PS girder, deck slab, and diaphragm, $M_{DC1} = 1820.65$ kip-ft. and this moment is applied on noncomposite section.

Similarly, the midspan dead load moment due to sidewalk and steel railing, $M_{DC2} = 316.76$ kip-ft. and this moment is applied on composite section.

Dead load stress on noncomposite section =

$$f_{Nb} = -\frac{M_{DC1}\times y_{Nb}}{I_N} = -\frac{(1{,}820.65\times12)\times(34.5)}{400{,}653} = -1.881 \text{ ksi (tension)}$$

TABLE 15.52 Live Load Demands at Midspan, "*d*" Distance from Support and 1/3th Point of the Span

Live Load	Midspan (Moment + I) per Lane (kip-ft.)	Midspan (Moment + I) per Girder (kip-ft.)	End (Shear + I) per Lane (kip)	End (Shear + I) per Girder (kip)	1/3 Span (Shear + I) per Lane (kip)	1/3 Span (Shear + I) per Girder (kip)
HL93	2928.71	2081.11	111.66	88.31	66.06	52.25
Routine permit P5 truck	3235.32	2298.98	127.11	100.53	74.55	58.96
Special permit P13 truck	4697.07	1950.60	187.08	103.93	88.81	49.34

Dead load stress on composite section =

$$f_{CD} = -\frac{M_{DC2} \times y_{Cb}}{I_C} = -\frac{(316.76 \times 12) \times (50.301)}{840{,}416} = -0.228 \text{ ksi} (\text{tension})$$

Stress from prestress force =

$$f_{pr} = \frac{P_{se}}{A_N} + \frac{P_{se} \times e \times y_{Nb}}{I_N} = \frac{720}{684} + \frac{(720) \times (28.5) \times (34.5)}{400{,}653} = 2.820 \text{ ksi} (\text{compression})$$

Live load (HL93) moment per girder, M_{LL+I} = 2081.11 kip-ft.

Live load stress on composite section =

$$f_{LL+I} = -\frac{M_{LL+I} \times y_{Cb}}{I_C} = -\frac{(2{,}081.11 \times 12) \times (50.301)}{840{,}416} = -1.495 \text{ ksi} (\text{tension})$$

Inventory rating with AASHTO LRFD Service III (full dead load plus 80% live load) =

$$\frac{-0.424 \text{ ksi} - (f_{pr} + f_{Nb} + f_{CD})}{0.8 \times f_{LL+I}} = \frac{-0.424 - (2.820 - 1.881 - 0.228)}{0.8 \cdot (-1.495)} = 0.95$$

Operating rating with AASHTO LRFD Service III is not required according to MBE.

For Service I load rating for routine P5 permit truck, the stress at the top of slab under permanent plus transient loads can be determined as follows:

Allowable compressive stress = 0.6 f_c' of slab = (0.6)(3 ksi) = 1.8 ksi

Dead load stress on composite section =

$$f_{Ct} = \frac{M_{DC2} \times y_{Ct}}{I_C} = \frac{(316.76 \times 12) \times (27 \times 824)}{840{,}416} = 0.126 \text{ ksi} (\text{compression})$$

Live load (P5) moment per girder, M_{LL+I} = 2298.98 kip-ft.

Live load stress on composite section =

$$f_{LL+I} = \frac{M_{LL+I} \times y_{Ct}}{I_C} = \frac{(2{,}298.98 \times 12) \times (27.824)}{840{,}416} = 0.913 \text{ ksi} (\text{compression})$$

Load rating with AASHTO LRFD Service I = $\dfrac{1.8 \text{ ksi} - f_{Ct}}{f_{LL+I}} = \dfrac{1.8 - 0.126}{0.913} = 1.83$

Similarly,

Live load (P13) moment per girder, M_{LL+I} = 1950.60 kip-ft.

Live load stress on composite section = f_{LL+I} = 0.775 ksi (compression)

Load rating with AASHTO LRFD Service I = 2.16

5. Moment Capacity at Midspan and the Corresponding Ratings at Ultimate Level

The moment capacity can be calculated with the following formula:

$$M_n = A_{ps} \times f_{ps} \times \left(d_p - \frac{a}{2} \right)$$

where A_{ps} = area of prestressing steel, f_{ps} = average stress in prestressing steel at nominal bending resistance, d_p = distance from extreme compression fiber to the centroid of prestressing tendons, M_n = nominal flexural resistance, f_{pu} = specified tensile strength of prestressing steel.

$$f_{ps} = f_{pu} \times \left(1 - k \times \frac{c}{d_p}\right)$$

$$a = \beta_1 \times c$$

$$c = \frac{A_{ps} \times f_{pu}}{0.85 \times f_c' \times \beta_1 \times b + k \times A_{ps} \times \dfrac{f_{pu}}{d_p}}$$

$k = 0.38$ for stress-relieved strand (LRFD Specifications Table C5.7.3.1.1-1)

$$c = \frac{(4.675)\times(270)}{0.85\times(3)\times(0.85)\times(92)+(0.38)\times(4.675)\times\dfrac{270}{72.125}} = 6.126 \text{ in.}$$

$$f_{ps} = (270)\times\left(1-0.38\times\frac{6.126}{72.125}\right) = 261.29 \text{ ksi}; \; a = (0.85)\times(6.126) = 5.207 \text{ in.}$$

$$M_n = (4.675)\times(261.29)\times\left(72.125-\frac{5.207}{2}\right)\times\frac{1}{12} = 7076.8 \text{ kip-ft.}; \varphi = 1.0$$

To determine the final value of ultimate moment, the cracking moment, M^*_{cr} should be determined.

Modulus of rupture of normal weight concrete = $f_r = 7.5\times\sqrt{f_c'} = 7.5\times\sqrt{5000} = 530$ psi.

Noncomposite dead load moment, M_{DC1} = 1,820.65 kip-ft.
Compression stress in concrete due to effective prestress forces only =

$$f_p = \frac{P_{se}}{A_C} + \frac{P_{se}\times e\times y_{Cb}}{I_C} = \frac{720}{1{,}120.5} + \frac{(720)\times(50.301-6)\times(50.301)}{840{,}416} = 2.55 \text{ ksi}$$

Noncomposite section modulus = $S_N = \dfrac{I_N}{y_{Nb}} = \dfrac{400{,}653}{34.5} = 11{,}613$ in

Composite section modulus = $S_C = \dfrac{I_C}{y_{Cb}} = \dfrac{840{,}416}{50.301} = 16{,}708 \text{ in.}^3$

$$M^*_{cr} = (f_r + f_p)\times S_N - M_{DC1}\times\left(\frac{S_C}{S_N}-1\right) = \frac{(0.53+2.55)\times(11{,}613)}{12} - (1{,}820.65)\times\left(\frac{16{,}708}{11{,}613}-1\right) = 2{,}182 \text{ kip-ft.}$$

Therefore, the ultimate moment capacity = max(φMn, 1.2 M^*_{cr}) = 7,076.8 kip-ft.
The corresponding ratings are then calculated and summarized in Table 15.53.

6. Shear Capacity at Support End (62 ½ in. from Support) and the Corresponding Ratings

Shear capacity is given by Equations 5.8.3.3-1 through 5.8.3.3-4 of LRFD and are listed in the following. The nominal shear resistance, V_n, shall be the lesser than

$$\begin{cases} V_n = V_c + V_s + V_p \\ V_n = 0.25\times f_c'\times b_v\times d_v + V_p \end{cases}$$

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TABLE 15.53 Ultimate Moment Rating Calculations at Midspan

Strength I limit state	HL93 (inventory) rating	$\frac{7076.8-1.25\times 2137.4}{1.75\times 2081.11}=1.21$
Strength I limit state	HL93 (operating) rating	$\frac{7076.8-1.25\times 2137.4}{1.35\times 2081.11}=1.57$
Strength II limit state	Routine permit P5 rating	$\frac{7076.8-1.25\times 2137.4}{1.58\times 2298.98}=1.21$
Strength II limit state	Special permit P13 rating	$\frac{7076.8-1.25\times 2137.4}{1.50\times 1950.60}=1.51$

TABLE 15.54 Final Ultimate Shear Capacity Based on Iteration Performed at the Location of Support End

Load Combination	Assumed RF	θ (degree)	β	V_c (kip)	V_s (kip)	V_p (kip)	V_n (kip)	Estimated RF
HL93 inventory	1.96	26.72	2.533	70.47	317.00	52.52	440.00	1.96
HL93 operating	2.54	26.72	2.533	70.47	317.00	52.52	440.00	2.54
Routine P5	1.92	26.53	2.538	70.61	319.64	52.52	442.77	1.92
Special P13	1.96	26.53	2.538	70.61	319.64	52.52	442.77	1.96

Note: The values of ε_x for all above cases are negative.

where V_n = nominal shear resistance of the section considered, V_c = nominal shear resistance provided by tensile stresses in the concrete, V_s = shear resistance provided by shear reinforcement, V_p = component in the direction of the applied shear of the effective prestressing force; positive if resisting the applied shear, b_v = width of prestressed I-girder web, d_v = effective shear depth.

$$V_c = 0.0316\times\beta\times\sqrt{f_c'}\times b_v\times d_v;\ V_s = \frac{A_v\times f_y\times d_v\times\cot(\theta)}{s}$$

$$\text{The inclination of the strand} = \tan^{-1}\left(\frac{36-6\text{ in.}}{1230.5\text{ in.}/3}\right) = 0.073\text{ radian}$$

The vertical component of prestressing force, V_p = (720 kip) sin(0.073) = 52.52 kip.

For b_v = 7 in., d_v = 0.72h = (0.72)(78.125 in.) = 56.25 in. and the average stirrup spacing, s = 5.64 in., f_y = 40 ksi, and A_v = 0.4 in.^{2}, the iteration process determining the values of θ and β can then performed and the results are summarized in Table 15.54.

The corresponding rating factors for HL93 inventory, operating, routine P5, and special P5 are 1.96, 2.54, 1.92, and 1.96 respectively.

7. Shear Capacity at 1/3 Span Point and the Corresponding Ratings

Similar procedures shown in previous section are performed at the 1/3 span point. Few parameters that have been different from previous section are listed as follows:

$d_v = d_p - a/2$ = 72.125 − 5.207 in./2 = 69.52 in.

V_p = 0 as the angle of inclination of strand is equal to zero

The stirrup spacing, s, at 1/3 span point = 22 in.

The values of θ and β after iterations and the corresponding rating factors are listed in Table 15.55.

The corresponding rating factors for HL93 inventory, operating, routine P5, and special P5 are 1.08, 1.40, 1.22, and 1.54 respectively.

8. Summary

The ratings under different truck loadings are summarized in Table 15.56.

Critical ratings of the interior girder are 0.95 at inventory level and 1.40 at operating level for HL93 loading. Critical ratings for P5 truck and P13 truck are 1.21 and 1.51, respectively.

TABLE 15.55 Final Ultimate Shear Capacity Based on Iteration Performed at the Location of 1/3 Span Point

Load Combination	Assumed RF	θ (degree)	β	V_c (kip)	V_s (kip)	V_p (kip)	V_n (kip)	Estimated RF
HL93 inventory	1.08	35.15	2.299	79.05	71.81	0.00	150.86	1.08
HL93 operating	1.40	35.15	2.299	79.05	71.81	0.00	150.86	1.40
Routine P5	1.22	31.85	2.501	86.00	81.39	0.00	167.39	1.22
Special P13	1.54	31.70	2.512	86.38	81.86	0.00	168.24	1.54

Note: The values of ε_x for all above cases are positive.

TABLE 15.56 Summary of LRFR

	Service Level Stress		Ultimate Moment at Midspan	Ultimate Shear at Support End	Ultimate Shear at 1/3 Span Point
Rating	Service I	Service III	Strength I/Strength II	Strength I/Strength II	Strength I/Strength II
HL93 (inventory)	—	0.95	1.21	1.96	1.08
HL93 (operating)	—	—	1.57	2.54	1.40
Routine permit P5 truck	1.83	—	1.21	1.92	1.22
Special permit P13 truck	2.85	—	1.51	1.96	1.54

15.4 Summary

This chapter has presented the basic principles of load rating in general and discussed the important issues that should be considered when rating concrete bridge. Several examples of concrete bridges are presented in this chapter. As always, changes to the specification will affect the rating procedure; however, these examples will provide a basic principle of rating of concrete bridges.

References

AASHTO. 2002. *Standard Specification for Highway Bridges, 17th Edition*. American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. 2007. *LRFD Bridge Design Specifications, 4th Edition*. American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. 2012. *LRFD Bridge Design Specifications*, Customary U.S. Unit, 2012. American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. 2013. The Manual for Bridge Evaluation, 2nd Edition, with 2011 and 2013 Interim Revisions. American Association of State Highway and Transportation Officials, Washington, D.C.

16

Rehabilitation and Strengthening of Highway Bridge Superstructures

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16.1 Introduction

Currently there are approximately 605,000 bridges (over 20 ft. [6 m] in length) in all public highway systems in the United States. Approximately 23.8% of them are functionally obsolete and structurally deficient (FHWA 2012a). Bridges are classified as "Functionally Obsolete" (FO) when the deck geometry, clearance, or approach roadway alignment no longer meets current requirements and standards. FO can be classified as low load-carrying capacity, low waterway adequacy, insufficient deck roadway width (lane or shoulder width), insufficient horizontal and vertical clearances, and poor approach roadway alignment (FHWA 2011a) to serve traffic demands (vehicular size, traffic speed, traffic volume, etc.), or inadequate waterway to handle occasional flooding flows. Bridges are considered "Structurally Deficient" (SD) if significant load-carrying elements are found to be in poor condition due to deterioration and/or damage, or the adequacy of the waterway opening provided by the bridge is determined to be extremely insufficient to the point of causing overtopping with intolerable traffic interruptions (FHWA 2011a). In other words, the bridges have major deterioration, cracks, foundation scouring/undermining, or other damage and/or flaws that reduce their ability to safely support vehicle loads. SD bridges do not immediately imply that they will collapse or are unsafe, but need significant maintenance attention, such as repair, rehabilitation, retrofit, strengthening, or replacement (see Section 16.3.1 for definitions). The percentage of bridges falling into these two categories has gradually dropped down since 1992 as a result of new bridge constructions and, for the most part, replacement and rehabilitation (repair/retrofit) of SD bridges over the years. Among all the bridges nationwide, 33.4% have steel superstructures (including plate girders, box girders, and trusses). On the basis of the year 2004 National Bridge Inventory (NBI), the average age of the bridges is 40 years and that for steel bridges is 48 years. Among steel bridges, 36.7% are SD and FO, the percentage is higher than that for all bridges.

Figure 16.1 shows the relationship of construction year versus percentage of total bridges (all types of bridges on all highway systems), as well as versus percentage of SD bridges. The figure shows that most bridges were constructed after World War II. Of the total bridges, 11.2% are considered SD as of 2011. Similarly, Figure 16.2 shows the age versus percentage of total steel bridges and SD bridges among the total steel bridges (Cheng and Fisher 2006). It can be seen that there were two steel bridge construction peaks 65–75 and 35–55 years ago, that is, before World War II (in 1930 through 1940) and during the interstate construction era in the 1950s through 1970s. Among these steel bridges, as of today, 73.5% are over 40 years old, 55.7% over 50 years old, 37.1% over 60 years old, and 10.6% over 75 years old. Of all steel bridges, 17.9% are considered SD.

Because many bridges have not been adequately maintained and timely repaired, various types of deterioration or member failure in bridges have been reported. In the wake of increase of bridge service years, severe environmental factors (e.g., rain, snow, moisture, seawater, temperature change, deicing salt, airborne pollution), increase of traffic load and volume, and inappropriate use and inadequate maintenance, bridge deterioration could lead to a risk of unsafe service and member failures

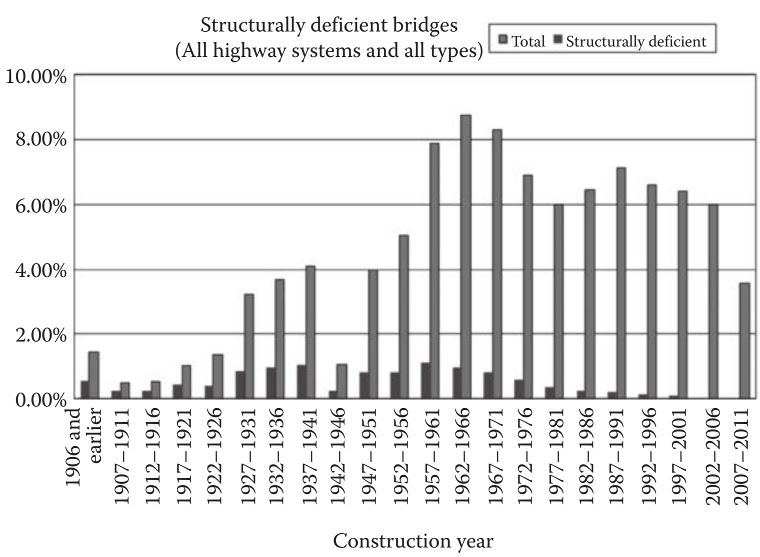


FIGURE 16.1 Year of construction versus rate of structurally deficient bridges as well as total bridges (as of 2011).

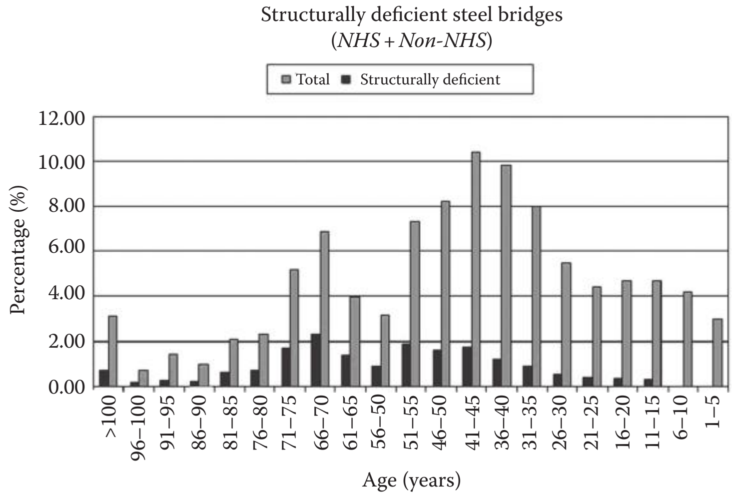


FIGURE 16.2 Age versus rate of structurally deficient steel bridges (as of 2004).

that would cause a loss of bridge member stiffness and load path, and eventually even a partial failure or entire bridge collapse. For example, fatigue is a type of major deterioration for steel bridges, and an excessive fatigue crack size may cause loss of strength or stiffness of primary members and thus fracture of a steel bridge. Typical deterioration of bridges includes member session loss because of corrosion, steel member fatigue damage/cracking, bearing lockup, concrete spalling/cracking, corrosion of steel reinforcement, scouring, connection rust, and so forth. Facing these growing problems, Federal Highway Administration (FHWA), American Association of State Highway and Transportation Officials (AASHTO), and bridge owners have fully realized the urgency and challenges, and have made significant efforts in bridge repair, restoration, rehabilitation, and replacement. Strengthening bridges of inadequate load-carrying capacity can be used as an alternative to replacement or posting of bridges.

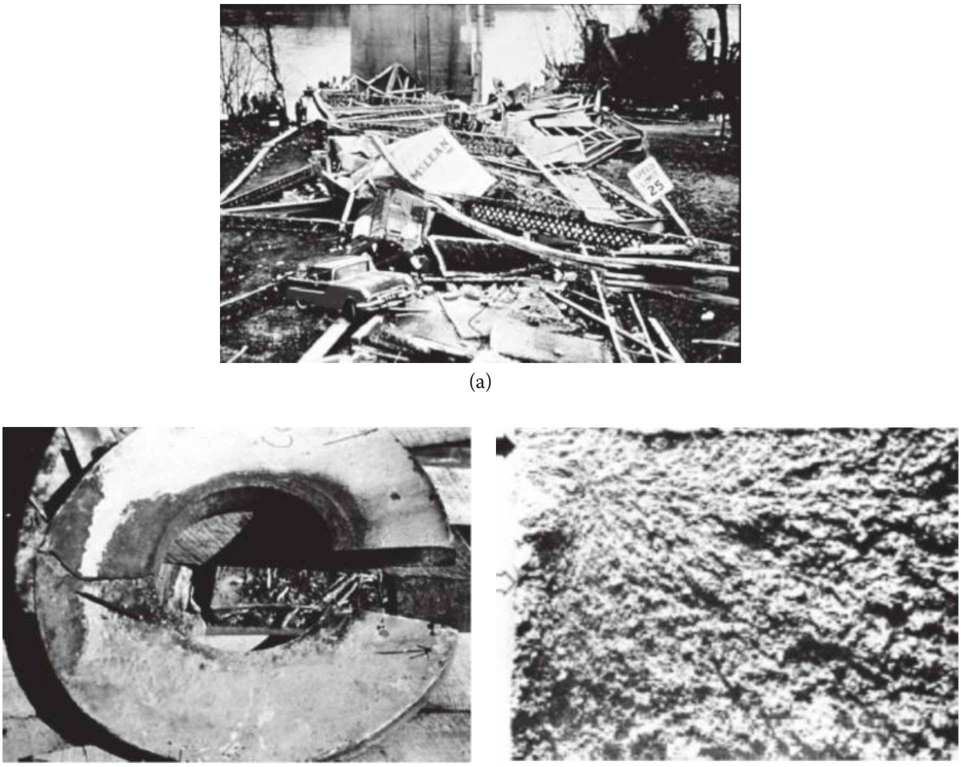
Having discussed bridge deterioration and conditions as earlier, this chapter is not going to discuss in detail on how to identify and assess bridge conditions. This chapter briefly discusses the types of various deteriorations/damages and other factors of bridges that demand rehabilitation and strengthening, several historical failures and lessons learned, definitions of bridge preservation terms, and methods and practical cases of rehabilitation and strengthening. The objectives of this chapter are to provide overall fundamentals and guidance of nonseismic rehabilitation and strengthening of highway bridge superstructures to engineering students and practicing engineers. For further detailed discussions, references may be made elsewhere (Shanafelt and Horn 1984, 1985; Silano 1993; Xanthakos 1995; Dorton and Reel 1997; Khan 2010; Kim 2011; Newman 2012). For seismic retrofit and strengthening, see Chapters 13 and 15 of *Bridge Engineering Handbook, Second Edition: Seismic Design*. For rehabilitation and strengthening of orthotropic steel decks, see Chapter 17.

16.2 Historical Collapses and Bridge Conditions

16.2.1 Steel Bridges

16.2.1.1 Collapse of the Silver Bridge

Although there have been numerous local or member failures that were caused by various deteriorations and critical connection fracture in steel bridges (such as Hoan Bridge, San Francisco–Oakland Bay Bridge, and Kentucky/Indiana I-64 Sherman Minton Bridge), only a few catastrophic fracture failures occurred to entire highway bridges in the United States (FHWA 1970, 1984; NTSB 2008).



(a)

(b)

(c)

FIGURE 16.3 Silver bridge collapse due to eyebar failure: (a) collapsed bridge, (b) cracked eyebar, (c) fractured section. (Courtesy of FHWA, 1970.)

The most horrific failure was the collapse of the U.S. Route 35 Silver Bridge over the Ohio River in Point Pleasant, West Virginia, killing 46 people on December 15, 1967 (Figure 16.3). The bridge had 700 ft. (213 m) center span and 380 ft. (116 m) side spans and suspension design with "eyebars" chained together instead of wire cables. The eyebars were linked together with massive pins. The initial failure was a cleavage fracture (brittle fracture) in the low limb of the eye at an eyebar joint and subsequently separating the eyebar from the chain, which caused the subsequent collapse of all bridge spans and towers. The Safety Board investigation found that the fracture was caused by the critical size cracking as a result of the combined action of stress corrosion, corrosion fatigue over nearly 40-year service life, and low toughness of steel at low temperature (30°F [–1°C]). Since then, the federal government has mandated the National Bridge Inspection Program, requiring periodic inspection of all nation's bridges, and launched the associated Highway Bridge Replacement and Rehabilitation Program (FHWA 2001) for the public safety. This catastrophic disaster and some other steel bridge fracture failures resulted in the final AASHTO Charpy V-Notch Impact requirements (or material toughness requirements) in 1976 as a fracture control plan on the main load-carrying components subjected to tensile stress in non-load-path-redundant bridges (AASHTO 1976).

16.2.1.2 Collapse of the Mianus River Bridge

Another catastrophic failure was the Mianus River Bridge on Interstate 95 over the Mianus River in Greenwich, Connecticut. The two-girder bridge collapsed and fell into the river, killing 3 people on June 28, 1983 (Figure 16.4). The 100 ft. (30 m) suspended span was attached to the adjacent cantilever spans by a "pin-and-hanger" (P-H) assembly at each of its four corners. The undetected displacement

FIGURE 16.4 Pin-and-hanger assembly failure of Mianus Bridge over I-95. (Courtesy of FHWA, 1984.)

of a loosened hanger from the pin, because of cumulated corrosion in the assembly, caused excessive forces and developed a fatigue crack on the surface of pin of the assembly that was broken. When this P-H assembly broke at one corner, the forces redistributed to the other corners, eventually causing the suspended span to fall down. The P-H connection had significant pack rust built-up behind the hanger and washer (between the washer and main member web), transmitting excessive force that moved the hanger toward the end of the pin because the retainer plate was too flexible. This resulted in a fatigue crack in the surface of the pin. After the pin cracked and broke, there was no longer any support. The hanger fell, and the corner of the bridge fell.

Following this event, significant research on fatigue of steel connections was performed, and tremendous insight into behavior of steel connections, including pin connections, riveted/bolted connections, and welded connections, was obtained. The program of inspection and maintenance was further enhanced through more rigorous inspection procedure for fracture-critical (FC) bridges.

16.2.1.3 Collapse of the I-35W Mississippi River Bridge

The most recent catastrophic failure was the collapse of I-35W truss bridge over Mississippi River in Minneapolis, Minnesota, on August 1, 2007, killing 13 people and injuring 145 people (NTSB 2008). The 1000 ft. (304.8 m) long deck truss had a main span of 456 ft. (139.0 m) with 108 ft. (32.9 m) into water. Figure 16.5 shows the main truss spans and node numbering. The National Transportation Safety Board (NTSB) investigation identified several major findings: (1) The collapse initiated with a lateral shifting instability of L9/U10W diagonal member at upper end and subsequently U10 node gusset plates (with preexisting buckling) failed; (2) The I-35W Bridge was non-load-path-redundant, so the total collapse of the deck truss was likely once the gusset plates failed; (3) The U10 node gusset plates had inadequate capacity for expected design loads, increased dead loads over the years and construction loads piled on the bridge deck when it collapsed. Other safety issues included insufficient quality control procedures of the bridge design firm; insufficient federal and state reviewing procedures; lack of guidance for placement of construction loads on the deck; exclusion or lack of regulations for design, inspection, and evaluation of gusset plate connections (NTSB 2008).

With the recommendations from the NTSB, the FHWA issued technical advisory on load-carrying capacity considerations of gusset plates of existing similar bridges and inspection using nondestructive evaluation technologies (FHWA 2008, 2010b). AASHTO also started requiring element-level inspection and load-rating evaluation for gusset plates (AASHTO 2013).

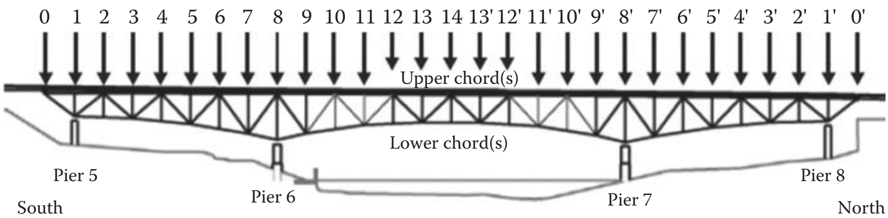


FIGURE 16.5 Main trusses of I-35W Bridge and node numbering. (Courtesy of NTSB, 2008.)

It should be noted that all the bridge collapses described earlier initiated from local distress of bridge elements/members and subsequently caused entire bridge collapse because of the loss of single load path in these non-load-path-redundant bridges. It is a very important concept in the modern bridge design that bridge load-path and system redundancy should be seriously considered.

16.2.1.4 Lessons Learned

For these steel bridges, the initial member/element failures could be attributed to the following:

1. Steel corrosion: Both Silver Bridge and Mianus Bridge were severely corroded at the eyebar connection and P-H connection.
2. Fatigue and fracture: Eyebar member in Silver Bridge initiated cracking under combined action of stress corrosion and corrosion fatigue, and brittle fracture occurred in the cold temperature with low toughness of the material.
3. Lack of load-carrying capacity: Several gusset plate connections in the I-35W Bridge were identified to be underdesigned for the design load and increased dead loads from bridge renovation (added deck thickness, medians, railings, deicing facilities, and so on) over the years.
4. Overload: When I-35W Bridge collapsed, there were a great deal of construction loads piled on the bridge deck, which overstressed and triggered the U10 gusset plate to continue bowing plastically until final failure.
5. Design/construction error or not functioning as originally designed: I-35W gusset plate connections were not appropriately designed and checked, resulting in insufficient load-carrying capacity since bridge opening. However, because of elastic material surrounding the plastic bowed area, it had survived until the time of bridge collapse. For the Mianus Bridge, because of severe corrosion cumulated in the P-H assembly, the P-H connection did not freely rotate as designed.
6. Connection failure: All three bridge failures initiated from member connections (eyebar, P-H, and gusset plates). Old design often assumed that connections had higher strength than the members they connected. However, errant design and nonstrength factors could cause failure that was not taken into account in the design. Connection design is a very important part of bridge design in terms of strength, serviceability, and fatigue.
7. Lack of system redundancy: Regardless of the causes of the bridge member failure, all the three bridges had a common feature, that is, the entire bridge collapsed because of the loss of the single load path (such as two-girder system) initiated from a local member/element failure, and the lack of redundancy of the bridge system. These types of bridges are called non-load-path-redundant bridges.

Local deterioration and damage are often seen in relatively old steel bridges because of poor design detailing and lack of maintenance effort. They may not result in entire bridge collapse if the bridge is redundant, nor result in immediate failure if the bridge is timely repaired, rehabilitated, and strengthened. It should be emphasized that bridge system redundancy is very critical in bridge design, inspection, and maintenance to maintain the integrity of the bridge system. Currently, it has been a common practice that requirements for inspection of FC bridges are more rigorous than normal bridges in inspection intervals, inspection procedures, qualifications of personnel, nondestructive testing (NDT), and records of inspection report and inventories.

16.2.2 Concrete Bridges

16.2.2.1 Collapse of the Koror–Babeldaob Bridge

The Koror–Babeldaob Bridge (Figure 16.6), with the world recorded segmental concrete box girder span of 240.8 m (790 ft.) at the time, suddenly collapsed on September 26, 1996, six months after the posttensioned concrete main span was retrofitted to correct excessive midspan deflection of more than 1.22 m (4 ft.) by installing additional prestressing and eliminating the midspan hinge (Klein 2008). The bridge connecting two main islands of the Palau was completed in 1977. The bridge superstructure of a two-lane single-cell box girder was built using balanced cantilever cast-in-place prestressed concrete box segments and a permanent midspan hinge. The collapse was triggered by compression-induced delamination of the top flange. A retrofit of installing additional prestressing to correct midspan sag increased the compressive stress in the top flange. The closure of the center hinge resulted in daily variations in the stress and the main span of the bridge collapsed in the Toagel Channel (Klein 2008). The retrofit and restrained thermal expansion substantially increased the compressive stress in the top flange over the main piers and the top flange was vulnerable to delamination because transverse reinforcement was not provided.

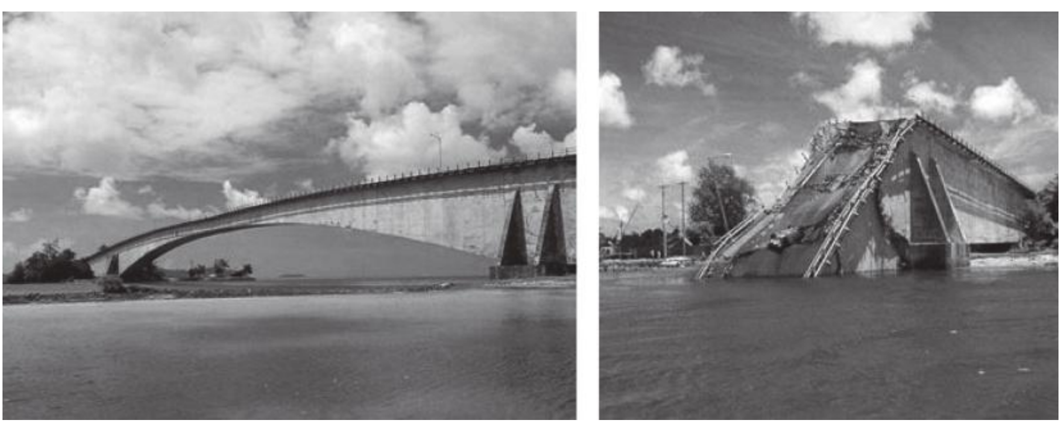
16.2.2.2 Collapse of the Lowe's Motor Speedway Pedestrian Bridge

On May 20, 2000, the Lowe's Motor Speedway pedestrian bridge in Concord, North Carolina, collapsed injuring 107 people (CNN 2000). This privately owned 5-year-old pedestrian bridge was a four-span, simply-supported, precast, pretensioned concrete bridge that spanned a major U.S. highway. The cause of the failure was the corrosion of prestressing steel cables within the bridge from calcium chloride in the grout that surrounded the cables. The amount of calcium chloride in the grout was so high that at the time of the collapse it had corroded the steel to less than one-fourth of its original size (Poston and West 2005; Alsup 2006).

16.2.2.3 Lessons Learned

The parties involved were subject to a confidentiality agreement, so no definitive statement has been made as to the cause of the collapse of the Koror–Babeldaob Bridge. The following lessons were learned from academic research and studies (Burgoyne and Scantlebury 2006, 2008; Bazant et al. 2010, 2011, 2012a,b):

- The failure was not caused by the retrofit, but unexpected flaws in the original design and construction were exposed by the retrofit.
- The construction industry should not shelter behind confidentiality clauses but, like the aircraft industry, publish its mistakes so lessons can be learned.



(a) (b)

FIGURE 16.6 Collapse of the Koror–Babeldaob Bridge. (Courtesy of OPAC–http://www.opacengineers.com/projects/koror, San Francisco, CA.)

- None of the available existing creep and shrinkage models is satisfactory as purely predictive tools, predicts significantly better results than others. Model B3 fits measured deflection closely using test data-based parameters. The inadequacy of the creep and shrinkage prediction from concrete strength and composition is the greatest source of error.
- The one-dimensional beam-type analysis results in great errors in deflections and prestress loss of the box girder. Three-dimensional shell analysis must be used to capture shear lags in slabs and webs because of dead load and prestressing tendons.
- For large, creep-sensitive structures, creep and shrinkage models must be updated by considering short-time tests of the creep and shrinkage of the concrete used in the project.
- Best successful design practice should be always followed to minimize deflections and prestress losses: (1) avoid midspan hinge, (2) use low long-time creep concrete, (3) select a prestressing tendon layout that minimizes deflections, (4) prestress at higher initial concrete strength, (5) use a proper higher level of prestress to produce an upward deflection, (6) use stiffer (deeper) girders, and (7) install empty ducts for possible later installation of additional tendons.

Lessons learned from the collapse of the Lowe's Motor Speedway pedestrian bridge are as follows:

- Failure was caused by the corrosion of the prestressing steel cables resulting from the presence of calcium chloride in the grout.
- Unapproved and banned concrete mixture should not be used.
- All private bridges shall be mandatorily inspected by government engineers to ensure the safety of travel on American bridges.

16.2.3 Typical Structural Deficiencies and Damages

16.2.3.1 Steel Bridges

Over the years, numerous bridge member deterioration and damage have been reported in regular or special bridge inspections. To investigate the general bridge conditions in the United States, the AASHTO/National Steel Bridge Alliance Collaboration Task Group 14 (TG-14) on "Field Repair and Retrofit of Existing Steel Bridges" recently conducted a nationwide survey with the steel bridge owners. The objectives of TG-14 survey were to document the typical conditions of existing steel bridges and provide practical guideline on typical repairs, retrofit, rehabilitation, and strengthening. The survey questionnaire on Conditions of Bridges or Bridge Elements included 10 items of conditions and several questions regarding these items. The 10 conditions included: (1) Fatigue and Fracture, (2) Corrosion, (3) Rivet/Bolt Deterioration and Weld Deterioration, (4) Low Load Capacity, (5) Non-Redundancy, (6) Impact Damage, (7) Fire Damage, (8) Bearing Deterioration, (9) Truss Bridge Gusset Plate Deterioration and Eyebar Deterioration, and (10) Others.

As a preliminary result of the responses from more than a dozen agencies, the top five severe deterioration conditions for steel bridges were: (1) Corrosion, (2) Bearings Deterioration, (3) Low Load Capacity, (4) Impact Damage, and (5) Deterioration at Rivet/Bolt Connections or Gusset Plate. The response statistics indicated that corrosion is the most frequent and severe problem in steel bridges (except reinforced concrete deck deterioration). Other deterioration frequently occurred at bearings, rivets/bolted joints, and gusset plates. However, they are also corrosion related. The condition for fatigue and fracture is not as frequent as expected, compared to other issues such as low load-carrying capacity and impact damage. Nonredundancy issue is relatively rare but critical.

It is noted that the size of the samples may not be sufficient to make conclusions, but it provides a general tendency of steel bridge conditions in the United States.

16.2.3.2 Concrete Bridges

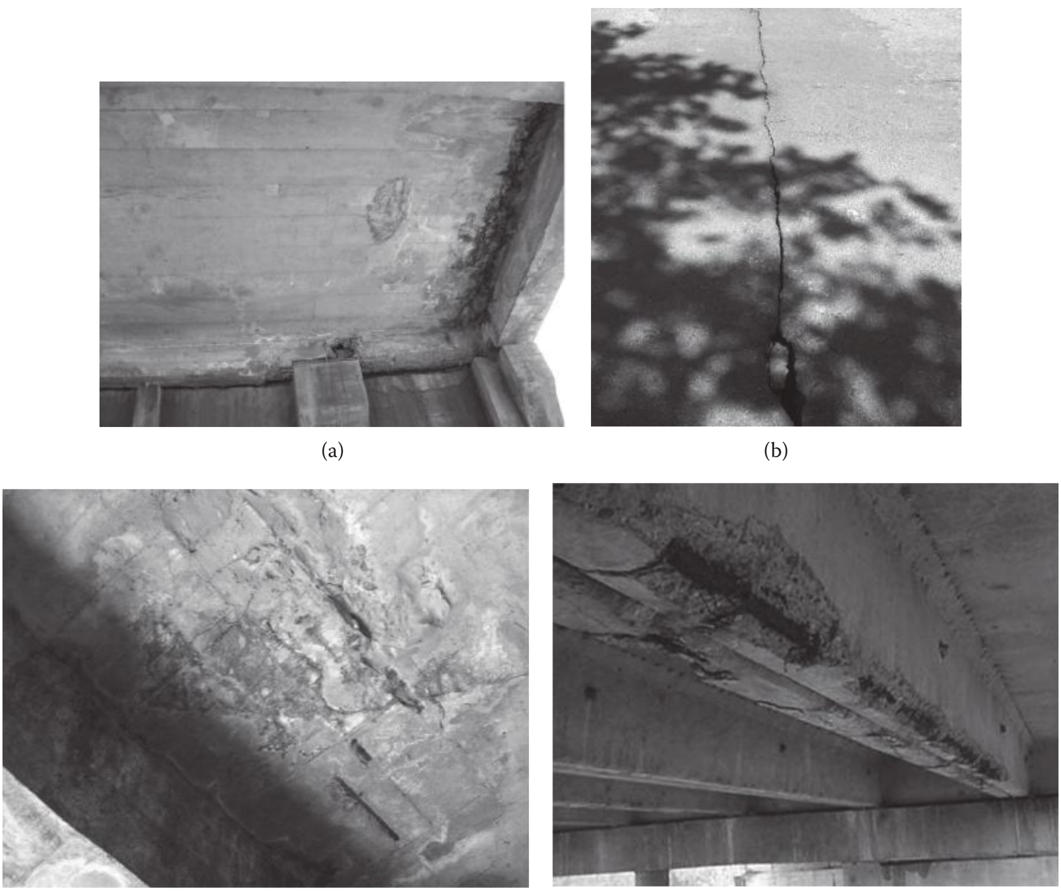
Concrete bridge deterioration is one of the leading causes of highway structural deficiency and possibly found in two forms: (1) concrete distress, and (2) reinforcement and prestressing tendon corrosion.

Typical damages to reinforced concrete (RC) and prestressed concrete (PC) bridge superstructure, especially to their girders, are as following:

- Leaks in expansion joints deteriorated concrete
- Cracks in various types (vertical, transverse, diagonal) and locations such as bottom flange, web, anchorage zones, and concrete decks
- Spalls of concrete covers

Cracks of various types are the most characteristic feature of damages in concrete structures. In general, crack width larger than 0.2 mm indicates certain harmful effects occurring both during construction and in service, such as insufficient vibration during casting, too many reinforcing bars in cross section, overloading, and corrosion of steel reinforcement (Radomski 2002).

Cracks in concrete decks are caused by numerous reasons such as cement mortar shrinkage, freeze–thaw cycles, settlement, and traffic loading. These cracks propagate through the deck depth, allowing rapid ingress of moisture and chloride ions into concrete interior leading to excessive deterioration because of rebar corrosion (Rahim et al. 2006). Figure 16.7 illustrates some typical damages for concrete bridge superstructure.



(a) (b) (c) (d)

FIGURE 16.7 Damages for concrete bridge superstructures: (a) bottom deck cracking corrosion, (b) top deck cracking, (c) exposed rebar, (d) reinforcement corrosion.

Cracks in prestressed concrete girders can be traced to one or several of the following sources (Podolny 1985):

- Inadequate provision for moment and shear capacities
- Improper detailing and poor standards
- Improper allowance for thermal force
- Introduction of new forces, such as heavy trucks, not provided for in the design
- Use of improper techniques at the time of the fabrication
- Poor tolerance in locating strands
- Poor workmanship, including substandard strengths of materials
- Introduction of excessive forces during the transportation of precast units
- Excessive erection forces
- Corrosion of prestressed tendons

16.3 General Guidelines and Considerations

16.3.1 Definitions

There are several frequently used terms that describe the maintenance work of existing bridges for restoring their functionality, elongating their service life, and increasing load-carrying capacity of bridge structures. Although there are no exact definitions, some of the definitions were provided by Klaiber et al. (1987, 2000) and are updated herein by referencing other documents (FHWA 2011a).

Maintenance: The technical aspect of the upkeep of the bridges; it is preventative in nature. Maintenance is the work required to keep a bridge in its present condition and to control and retard potential future deterioration.

Rehabilitation: The major process of restoring the bridge to its original service level or the original capacity to meet typical service loads and conditions experienced by the structure. It may add extended service life and functionality to the original bridge structure.

Repair: The technical aspect of rehabilitation; actions/activities taken to correct damage, or deterioration, or design/construction errors on a structure or element to restore it to its original condition.

Retrofit: Retrofit involves strengthening and upgrading the structure to meet loads or conditions that were not originally designed to experience.

Replacement: Total replacement of a bridge with a new facility constructed in the same general traffic corridor that must meet current geometric, construction, and structural standards required for the types and volume of projected traffic and loads over its design life.

Stiffening: Any technique that improves the in-service performance of an existing structure and thereby eliminates inadequacies in serviceability (such as excessive deflections, excessive cracking, or unacceptable vibrations).

Strengthening: The increase of the load-carrying capacity of an existing structure by providing the structure with a service level higher than the structure originally had (sometimes referred to as upgrading).

Sometimes, the terms *Restoration, Renovation, Reconstruction* are also used, and they are interrelated to the terms mentioned above.

It becomes a normal practice that when a bridge or a bridge member is determined to be repaired, rehabilitated, or replaced for a certain reason the bridge should be assessed for other aspects and other members, so that the maintenance work can be done at the same time with a reasonable budget, such as deck drainage, deck joints, deck patching, bearings cleaning, and railing and barriers. Also, results of refined analysis such as finite element method (FEM) and field testing may be used to avoid or limit the need of strengthening and rehabilitation work if they can verify the bridge safety and satisfactory performance.

In the past decades, the National Cooperative Highway Research Program (NCHRP) has sponsored several research projects on bridge repair, rehabilitation, retrofitting, and strengthening. Other public or private sections compiled documents for bridge repair and rehabilitation guidance (such as Silano 1993; Reid et al. 2001; Waheed et al. 2004; NJDOT 2007). These investigation reports and documents are excellent reference to the engineers.

The following sections discuss rehabilitation and strengthening of different bridge members and methods or technologies to be used.

16.3.2 Major Causes of Deterioration and Unsafe Conditions That Need Rehabilitation and Strengthening

Because severe deterioration and damage to bridge would cause bridge to be unsafe for service, sometimes even in jeopardy of failure, when deterioration and unsafe conditions have developed to a certain extent, the bridge engineer should consider rehabilitation or strengthening.

The reasons and lessons learned from bridge failures have been outlined in Section 16.2. Basically, possible reasons for rehabilitation and strengthening are similar in different countries (Reid et al. 2001), and they are as follows:

- Underdesign
- Faulty fabrication and construction
- More stringent design requirements since original design
- Deterioration because of corrosion, fatigue, cracking, and concrete spalling
- Impact damage
- Heavier loads since original design

Reasons and considerations for steel and concrete bridge strengthening and rehabilitation are outlined as follows:

1. Deterioration and damage:
   - Cross section loss of bridge main members
   - Fatigue cracks or fatigue-prone details of steel main members
   - Concrete cracking and rebar corrosion
2. Load-induced causes:
   - Significant increase of truck traffic volume and heavy truck overweight
   - Significantly added dead load, live load, and impact load because of widening or renovation, such as adding parapet/median barriers, and deck surfacing/thickening
   - Seismic loads
3. Environment-induced causes: Temperature, chloride, moisture
4. Material-induced causes: Deformation, creep, shrinkage, coating, curing
5. Design and/or construction errors: Underdesign (or unsatisfactory load rating), initial imperfections, or out-of-tolerances at the time of construction
6. Truck impacts and fire accidents: Many bridges built in the past have low underclearance and suffered damage by impact of vehicles. This often involves distortion of bottom portion of main girders (particularly fascia girders), causing the bridge to be only of partial use or entire closure. Impact usually occurs in central part structure spans where vehicle lanes are located under the bridge, and girder flange is still of partial effective positive bending capacity. If impact occurs in the negative moment zone, the damaged section of a steel girder may be critical to buckle under compressive stresses. For steel girders, the welds between the bottom flange and the web may be cracking under severe distortion, which requires postimpact inspection by nondestructive evaluation (NDE) test (ultrasound test [UT] or magnetic particle test [MT]).

For a fire accident, steel bridge girders are most likely to be buckled under high fire temperature and the action of dead load on the bridge. In an extreme case, the MacArthur Maze interchange located near the eastern end of the San Francisco–Oakland Bay Bridge in Oakland, California, collapsed after a gasoline tanker truck fire (*New York Times* 2007) and eventually the entire superstructure was replaced.

7. Security upgrade: Man-made hazards should also be considered when a bridge is deemed to be a critical bridge because of the location, traffic condition, consequence of failure, and so on, due to probability of the impact or explosion to any part of bridges. The bridge should be assessed for vulnerability and impact loads, analyzed and redesigned to upgrade the load-carrying capacity based on the requirements of bridge importance and the postevent service level. Proactive measures are always considered better, such as to restrict the access to the bridge, and install surveillance monitoring equipments. Examples of structural upgrade include: to prohibit the use of non-load-path-redundant bridge members; to protect all main load-carrying members from direct impact from automobile, marine, or rail traffic; and to analyze impact load and strengthen the bridge members when these protection measures are not realistic.

16.3.3 Evaluation of Bridge Conditions for Rehabilitation and Strengthening

The bridge conditions should be inspected, measured, evaluated/load rated, and analyzed so that a decision can be made on whether the bridge should be rehabilitated, strengthened, or replaced with prompt actions or in lower priorities. In the United States, there is a complete system for NBI, National Bridge Inspection Standards (NBIS), and bridge condition evaluation and load rating (AASHTO 2013). Readers may refer to Chapters 11 through 15.

In many cases, to avoid or limit strengthening/rehabilitation actions, accurate assessment may be used when all bridge conditions, imperfections/defects, and bridge member/element sizes are known. In this case, bridge deterioration and any defects have to be taken into account in the assessment. NDT should be used to measure the defects, including initial imperfections, corrosion section loss, weld defects and fatigue cracks, collision damage, and excessive deflection/distortion. Stresses subjected by the bridge can be analyzed through empirical or refined analysis (such as FEM), or directly measured by field load testing with strain gages under action of known testing trucks and real traffic. On the basis of the results, engineers can make a decision, develop action plans, and execute rehabilitation and strengthening work with adequate confidence.

16.3.4 Decision-Making Considerations

Many factors should be taken into account when making decisions for major rehabilitation and strengthening projects. Generally rehabilitation and strengthening works are highly labor intensive and costly particularly requiring service traffic to be maintained. At this time, except standard repair, rehabilitation, and replacement of concrete decks (such as NJDOT 2009), there is no explicit guidance to make decisions for strengthening/replacement of bridge members or an entire bridge. In most cases, the decisions are made on project-to-project basis depending on the factors to be considered as follows.

16.3.4.1 Existing Conditions and Residual Service Years

In the United States, bridge inspection and evaluation follows FHWA and AASHTO requirements, as discussed in Chapters 13 through 15. Because of restriction of funding, site access, traffic flow, and environment condition, deteriorated bridges often remain in service even though they have overpassed the original design life. The bridge owner has to determine the extension of bridge service. If a bridge is decided to be replaced in a short period, it is not worth making major rehabilitation. However, evaluation is demanded to ensure bridge safety and integrity before the bridge is to be replaced. On the contrary, if a bridge is located in metropolitan area and difficult to replace, the bridge has to be rehabilitated to extend another cycle of design life. Many bridges such as in New York City and the State of New Jersey that have poor deterioration and have already severed for over a hundred years fall in this category.

16.3.4.2 Costs

Besides material costs, other factors may be significantly higher and should be considered when selecting optimal project methods to minimize the project costs as practical as possible. These factors include traffic management, working zone safety control, user cost because of lane closure and traffic congestion/delay, labor safety because of confined space on the bridge, health hazard, field equipment, and field work difficulty. Sometimes, for a large project, value engineering solutions are usually introduced.

16.3.4.3 Constructability

There are many possible methods for rehabilitation and strengthening. Some are more straightforward than others depending on the bridge location (over traffic, over water, or no interruption to anything), project time (bird/fish season, summer or winter, day or night), and bridge type. If the work is difficult to implement and workers are prone to dangerous condition, the contractor will face a risk of project delay. The project design should be practical and workable.

16.3.4.4 Safety

Of course the to-be-strengthened bridge structure itself should be designed to be safe after strengthening. Usually the cheaper option will be to target the weakness directly, such as web plates for shear, flanges for bending, and stiffeners for buckling. In the case that weakness area is difficult to access, alternative locations or alternative methods can be considered to relieve the weakness stress, for example, using external posttensioning prestress to increase live load-carrying capacity, or adding a support to reduce span length if allowed.

During construction, safety of both public and workers should be ensured. Motorists passing by or underneath the project site should not be put at risk during rehabilitation and strengthening work. Temporary barriers, fencing, falsework, and screen nets should be used to protect traffic and catch falling items.

16.3.4.5 Long-Term Maintenance

Rehabilitation and strengthening design should consider future maintenance burden and access. For example, field welds to repair cracking or holes should be avoided because field welds normally are difficult to control and the quality is poor so fatigue damage is easy to reoccur and repeated repairs may be required. Particularly, some corner locations are difficult to access, and thus hard to inspect and paint in the future.

16.4 Methods of Rehabilitation for Superstructure Members

16.4.1 Concrete Decks

16.4.1.1 Deck Repairs

In the United States, concrete deck deterioration is the topmost cause of bridge deteriorations and has low condition rating, no matter the deck surface is overlaid or not. Because bridge decks are subjected to direct traffic loads, especially truck loads, and deicing salt and rain/snow water, they are most severely deteriorated. Many times, maintenance engineers have to frequently repair pot holes, spalling/delamination, cracking, and efflorescence in concrete decks. Most commonly used type of bridge decks is reinforced concrete deck because of construction simplicity and composite effect with bridge girders. Depending on the depth of deterioration and corrosion, the following typical concrete deck repair procedures are normally adopted, as shown in Figure 16.8 (NJDOT 2007).

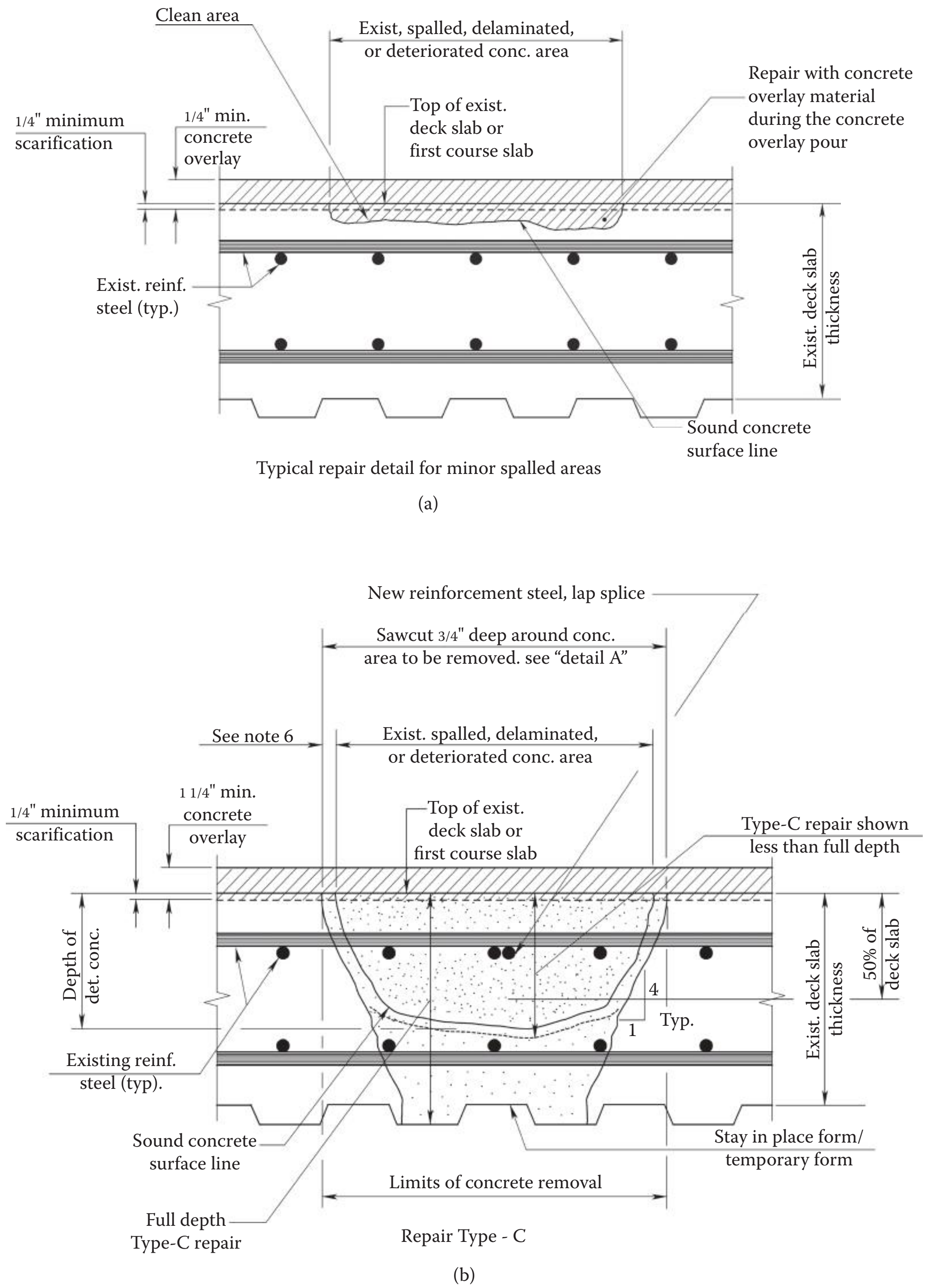
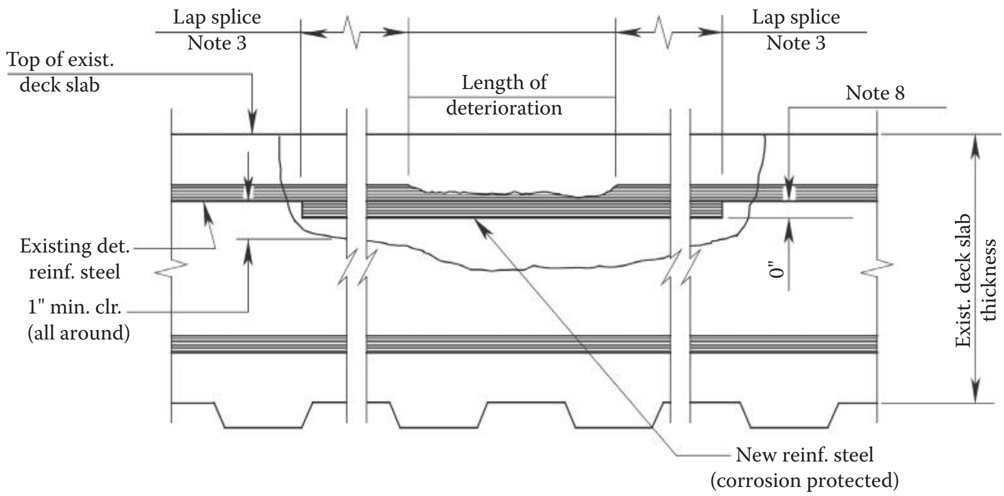
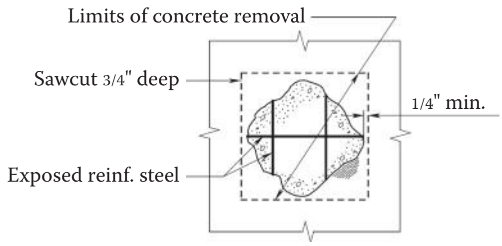


FIGURE 16.8 Concrete deck repairs: (a) shallow repair and (b) deep or total repair.



Deteriorated reinforcement steel repair plan

(c)



Plan view

(d)

FIGURE 16.8 (*Continued*) Concrete deck repairs: (c) rebar repair and (d) limits of repair area.

16.4.1.1.1 Shallow Repair

Where the depth of concrete deterioration is less than the concrete cover and reinforcing steel is not exposed, a shallow repair is needed. This is also called patching. The deteriorated concrete is saw-cut and removed by a pneumatic hammer or by hydrodemolition. The surface should be cleaned and repair material (normally nonshrinkage quick-setting polymer-modified cementations mortar) is applied and cured.

16.4.1.1.2 Deep Repair

Where deteriorated concrete reaches or is deeper than the top reinforcing steel, a deep repair is needed. At least 1 inch of concrete below the top rebars is removed. The rebars are cleaned by sandblasting or hydrodemolition, and additional bars are added for section losses (corroded or broken rebars). Epoxy bonding coat is applied to the rebars and quick-setting repair material or concrete (for large quantities) is applied and cured.

Where concrete deterioration depth extends to greater than 50% of the deck thickness, deep repair is needed unless the bottom rebars are not exposed. Otherwise, deck replacement in this portion is needed. If stay-in-place steel forms are not shown, a temporary form and shielding shall be needed to cast concrete and protect traffic or waterway under the bridge from falling debris.

16.4.1.1.3 Deck Overlay

Deck overlay or wearing surface has two functions: (1) sealing and preventing water and deicing salts from penetrating into the deck slab, and (2) providing rideability with a smooth and skid-resistant surface to minimize vehicular traffic impact. The overlay can be rigid (concrete overlay) or flexible (asphalt overlay). Presently, most used three types of overlay include latex-modified concrete (LMC), asphalt overlay with water proofing membrane, and thin-layer polymer epoxy concrete overlay (slurry type).

16.4.1.2 Deck Replacement

Where the concrete deck deterioration is so significant that deck repair is not available or uneconomical in the terms of labor cost, traffic control, and service life, an entire deck replacement may be required. On the other hand, reinforcement corrosion is a big issue because it is not visible by inspectors and hard to repair. Nowadays many NDT/NDE technologies have been rapidly developed and applied to the bridge deck inspection and condition assessment. These NDT tools can be used to monitor the deck condition for both concrete (top and bottom surfaces) and reinforcement materials. Once the deck condition is rated low/unsafe, a deck replacement is required.

Deck replacement is one of the most performed bridge rehabilitation and maintenance works. When a bridge superstructure or a bridge deck is determined to be replaced, new types of decks may be considered to reduce the bridge weight, increase corrosion resistance (i.e., increase service life), increase fatigue resistance or life, and enhance live load-carrying capacity. The weight of lightweight decks can be further reduced by using lightweight railing and barriers (Klaiber and Wipf 2000). For deck replacement, except normal concrete decks (cast-in-place or precast), the types of lightweight decks include: (1) open grid steel decks, (2) concrete-filled gird steel deck, (3) exodermic deck that looks similar to the concrete-filled grid decks in appearance, (4) lightweight concrete deck, (5) steel orthotropic deck, (6) aluminum orthotropic deck, (7) fiber-reinforced polymer (FRP) deck (1/5 concrete weight), and (8) laminated timber deck. However, not all lightweight decks are appropriate for the bridge with heavy truck traffic because of fatigue issue (such open grid deck, steel or aluminum orthotropic deck), and vibration or deflection issue (FRP deck). Engineers should take caution when selecting a type of bridge decks for bridge rehabilitation depending on the bridge service condition and the deck properties.

The practice of deck replacement or widening has been advanced to use high-performance concrete material, epoxy-coated or galvanized rebars, or even stainless steel rebars, and construction staging procedure to maintain traffic during construction. This will not be further discussed herein.

It is noted that during a rehabilitation project, a deck replacement can be carried out in conjunction with other rehabilitation/strengthening works. When an existing bridge deck has been removed, the bridge girders are exposed, the expansion joints can be easily replaced, the bearings can be jacked up readily, and particularly, composite action of steel bridge deck and girders can be added by installing shear connections, which will be discussed in Section 16.5.

16.4.2 Bearings and Joints

16.4.2.1 Bearings

Bearings are another top source of bridge deteriorations. Bearings transmit various vertical and horizontal forces from the superstructure and the substructure. Failure to properly accommodate bridge movement could lead to consequences of malfunction, service damage, or bridge collapse, such as overstress, fatigue, and seismic failure. The two main bridge components that provide relief from forces caused by movement from temperature and horizontal loads are bearings and deck joints. Bearings are designed to be "expansion" and "fixed," and should be functioning as designed throughout the design life. Whenever possible, design should minimize the number of bearings (and joints), such as using continuous spans and integral abutments, saving not only cost of construction but also cost of maintenance. All bearings are basically composed of a sole plate, a masonry plate, anchor bolts, and other components between the sole plate and masonry plate depending on the type of bearings.

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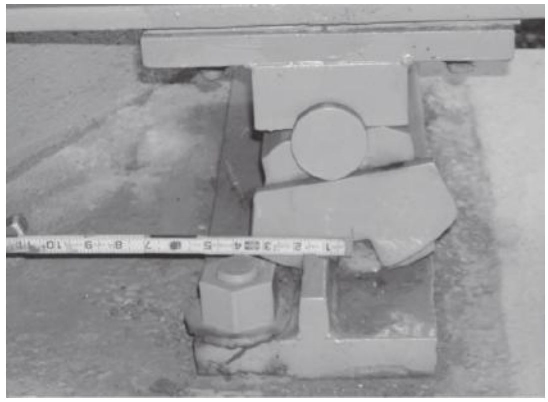
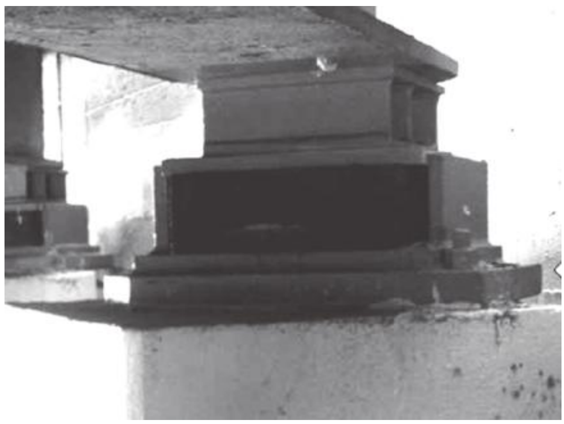
(a)



(b)



(c)

FIGURE 16.9 Old-style steel bearing deteriorations and replacement: (a) severe rust of steel rocker bearing, (b) rocker bearing creeping to one side (expansion), (c) replaced by elastomeric bearing.

The primary issues of bearing deteriorations include leakage of rainwater, deicing chemicals, and oil/grease from deck joints, as well as dirt, dust, grime, debris, and moisture accumulated from surroundings over the service years. These deteriorations may cause bearings to be "locked" or "frozen" from moving and rotating to accommodate temperature movement and live load action. Figure 16.9 shows an example of rust of an old-style steel rocker bearing (expansion). The pin of a new bearing should allow rotation, the curved rocker surface should allow pin to shift relative to the masonry plate on the concrete by itself, thus accommodating both expansion and contraction of the girder.

16.4.2.1.1 Cleaning, Painting, and Lubrication

Bearings that are corroded without major section loss can be cleaned and painted. It is more appropriate for moderate corrosion to remove the existing coating by blast cleaning along with removing debris, grime, and rust. The contact surfaces should be protected during cleaning and painting operations. Some owners use galvanized bearings and regalvanize them if severely corroded.

Pin and sole/masonry contact surfaces are generally lubricated as a maintenance item for steel bearings. Regular lubrication with a corrosion inhibitor would help prevent buildup of corrosion and grime, and help ensure contact surfaces are free to move. Lubricant materials include grease, lightweight oil, WD-40, and graphite-based compounds. Although lubricants are frequently used on steel interfaces, the constant intermittent leakage flushing from deck joints or moisture exposure will eventually remove the lubricants. To fully restore the bearing function, lubrication has to combine with correction of source of water, chemicals, and grime intrusions.

16.4.2.1.2 Replacement by Elastomeric Bearings

In the case that bearing is severely corroded and very difficult to repair/rehabilitate, or when bearing is no longer functioning as designed, they should be replaced. To avoid the similar corrosion and deterioration to the steel bearings, elastomeric bearings are recommended. However, their load-carrying capacity need to be assessed. Elastomeric bearings are more resistant to water and chemical attacks. This would also provide better performance in larger movement, such as truck vibration and seismic bridges in low-to-medium seismic regions when seismic loads are not a primary controlling factor of the bridge design. Figure 16.9 shows a replaced elastomeric bearing.

An elastomeric bearing is constructed of natural or synthetic elastomer pads and usually internally reinforced with steel shims. It relies on the distortion of the elastomer to provide for movement. This type of bearing is currently the most used in highway bridges. Some seismic elastomeric bearings include a lead core to resist significant horizontal loads.

When bearing is in repair and replacement, bridge girders need to be jacketed up onto temporary supports, which is labor intensive and time consuming, thus a major consideration should be given to the rehabilitation work if possible. Not all bridges were designed with replacement of bearings in mind. The load from the superstructure members is relieved from bearings when bearings need to be replaced. Strong points may be required for jacketing and must be effectively installed by installing suitable brackets, diaphragms, and stiffeners with adequate fit. If the old and new bearings are different in height and size, the concrete pedestal dimension needs to be adjusted. The detailed discussion on jacketing can be found elsewhere (Silano 1993).

It should be noted that bridge closure to traffic would be preferable for safety and time saving for construction as it permits entire girder line lift at a time. However, most of the time, particularly in metropolitan area where bridge closure is impossible, various jacking up methods have to be used depending on the site-specific scenario.

16.4.2.1.3 Anchor Bolt Strengthening or Replacement

Bridge inspection has often found anchor bolt deteriorations, such as corroded, bent, sheared off, cracked (by ultrasonic testing), and missing nuts or stripped threads. In such cases, anchor bolts repair or replacement is needed. In some cases, increase in load-carrying capacity by bridge strengthening or seismic retrofit also requires anchor bolt strengthening, such as by replacing anchor bolts, or by increasing the number and diameter of anchor bolts. Anchor bolt replacement can be done with or without removing the remainder of the bearing, by coring a hole through the masonry plate into the concrete bridge seat. A new bolt can be grouted into the place.

Cracks, spalls, or other concrete failure around the anchor bolts should be repaired and requires more extensive work to restore sound concrete around the anchor bolts and ensure anchor bolt embedment.

16.4.2.2 Joints

Similar to bridge bearings, deck joints (or expansion joints) are one of the important elements for bridges allowing girder expansion and contraction under temperature change, traffic load, seismic load, time-related movement (e.g., shrinkage, relaxation, and creep), and other loads. Their long-term durability and leakage protection is of great importance. The recent survey showed that it is one of the chief causes of bridge deteriorations. Figure 16.10 shows heavily rusted finger joint and an exposed steel plate on the deck over the joint. Severe deterioration may cause deck joints to be "frozen" and unable to move, so bridge elements may experience overstress and damage. Old design normally uses simple span, noncomposite construction with many joints because of simple design and construction. Modern design realized many joint-related problems, and thus new bridge and rehabilitation design should eliminate joints or minimize the number of joints as practical as possible, such as by using concepts of integral abutment

(a) (b)

FIGURE 16.10 Deterioration at expansion joints: (a) steel bridge joint, (b) concrete bridge joint.

bridges, continuous span, and making simple spans continuous at the joints. These concepts help reduce joint-related issues with leakage, truck ride impact, and concrete deck spalling/delamination around the joint, and thus enhance overall superstructure behavior.

16.4.2.2.1 Joint Types

There are various types of bridge joints. A fixed joint accommodates rotation only, whereas an expansion joint allows longitudinal, transverse, and rotation movements. Fixed joints include (1) open joint, (2) filled joint, and (3) compression joint, whereas expansion joints include (1) sliding plate joint, (2) finger plate, (3) saw-tooth plate joint, (4) strip seal joint, (5) sheet seal/plank seal joint, and (6) modular joint.

Presently the deck joints most commonly used in new bridges and replacements are as follows: (1) filled joint for small movements (1/2–2 in.) in short spans, (2) compression joints (preformed elastomeric joints) and strip joints for moderate movement (2–4 in.), and (3) modular joints for large movements over 4 in. Special joints used for seismic design to accommodate greater movement during earthquake are not discussed herein. Properly designed joint system should be removable and replaceable for joint maintenance, which is one of the most performed routine maintenance works.

16.4.2.2.2 Joint Deterioration and Rehabilitation

Deck joint deteriorations include severe rusting of steel elements, debris accumulation, water/oil leakage, restraint of free movement, fatigue cracks, loosening and damage of elements, concrete damage around the joint, and differential surface level on two sides of the joint. Minor deterioration may require repair of joint elements and spalled concrete, partial replacement of other elements (seal, armoring, anchorage, etc.), removal of debris, cleaning and painting. Major deteriorations that affect joint functions should be removed by joint replacement, which will involve bridge decks around the joint anchorage area.

16.4.2.2.3 Joint Replacement and Elimination

When deck rehabilitation or replacement is necessary, deck joints also need to be considered at the same time, such as joint replacement or elimination. The bridge joint rehabilitation, replacement, or elimination should be carefully considered with rehabilitation work of decks as a system, depending on the bridge type and scale. Particularly, when a deck joint is determined to be eliminated by making the bridge continuous at the original joint location, the bridge behavior will be changed and should be carefully reevaluated in rehabilitation design for strength and deformation capacity under expected temperature forces, traffic loads, and other loads. If the capacity is not sufficient and needs extensive strengthening or stiffening of other bridge members than the joint, obviously the elimination of joints is probably not appropriate for this case.

Further practical details and procedures for joint rehabilitation, replacement, and elimination can be found elsewhere (Silano 1993), as well as in Section 16.5.3.

16.4.3 Pin-and-Hanger Assemblies

Deterioration of P-H assemblies primarily includes corrosion, internal cracking in the pin, hanger cracking, and dust/rust/debris accumulation, which causes the pin "frozen" (not properly function), and other problems. The failure in the pin possibly cannot be detected with naked eyes, therefore NDT is required, such as UT, as shown in Figure 16.11.

P-H assemblies are normally used as supports of a suspended span (or dropped-in span) in steel bridges. This old detail design has been considered as a critical detail especially for two-girder bridge system since Mianus River Bridge collapse (see Section 16.2.1). The inspection of its damage and deterioration is very important. Most bridge owners require extensive FC inspection for P-H assemblies regardless of the number of girders of the bridge system. Figure 16.11 shows a severely corroded P-H assembly that caused it to be frozen and thus many fatigue cracks in the superstructure (Cheng et al. 2009). Replacement of the pin was performed, and the girder and assembly were repainted. A predrawn mark on the hanger surface has verified that the replaced pin is freely rotating and has restored the assembly function, as shown in Figure 16.11.

Because a steel bridge girder with a suspended span supported by P-H assemblies has lack of load-path redundancy, when a P-H fails, the bridge will partially or entirely fail depending on the number of girders or the bridge system redundancy. To prevent this from occurring, it is suggested to install a "catcher" at the P-H location to possibly catch the falling suspended girder in case the P-H connections fail and the girder falls. This is a very commonly used detail for such a type of steel girder bridges in the United States (Figure 16.11). However, the best method to retrofit and strengthen this type of bridges is to increase load-path redundancy of the bridge system, as discussed in Section 16.5.4.

16.4.4 Concrete Crack Repair

Main objective of concrete crack repair is to stop intrusion of water or chemicals, prevent seepage, recover structural integrity, and restore appearance of the member. For crack repair, epoxy grouts are usually used. Crack width ranging from 0.003 to 0.25 in. can be successfully injected with epoxy with varying viscosity. Cracks wider than 0.25 in. can be filled with either a cement grout or an epoxy grout with filler materials.

A crack wider than 0.02 in. and deeper than 12 in. from the horizontal plane can be filled by gravity. Narrower and shallower cracks require pressure injection. Injection pressure varies from 10 to 25 psi, depending on the resin viscosity and the width and depth of cracks. Common practice in bridges is to grout cracks wider than 0.0625 in. For a deck slab with overlay, cracks through deck depth are normally grouted by gravity. *ACI RAP Bulletin 2* (ACI 2003) provides the field guide to concrete crack repair procedures by gravity feed with resin.

(a) (b) (c) (d)

FIGURE 16.11 Pin-and-hanger assembly: (a) corrosion at pin-and-hanger assembly, (b) repainted pin-and-hanger assembly, (c) ultrasonic test of pin-and-hanger assembly, (d) catcher installed at pin-and-hanger assembly.

16.4.5 Heat Straightening of Steel Bridges

Low-clearance vehicular impact is one of the chief causes of bridge damages. Fire accident also causes steel bridge damage/buckling and even collapse, as discussed in Section 16.3.2. Carefully filled survey should be carried out for the distorted bridge members to determine extent and amount of distortion so that residual strength can be assessed and repair/strengthening design can be worked out. Field measurement test is another method to assess the load-carrying capacity. Depending on criticality of the damage and overall conditions of the bridge, the bridge may be repaired or replaced. For steel bridges, minor distortion or damage may be repaired by heat straightening, or adding cover plates. Figure 16.12 shows impact damages of a steel bridge girder and a concrete bridge girder. If stiffeners of a steel girder severely distort, shear and flexural resistances will be significantly affected. Similarly, when truck fire occurs, high temperature will cause steel bridge to yield and severely buckle/distort. Heat strengthening is a complex procedure for repair buckled/distorted steel girders by introducing heat to allow steel plastic deformation and strengthening the buckled material. It involves girder expansion and contraction, and caution should be taken to avoid overly residual stresses, brittle cracking, and brittle fracture through jacking over or repetitive repairs. A typical heat-strengthening sequence is shown in Figure 16.12. Details about types of impact damage, inspection and assessment of damages,

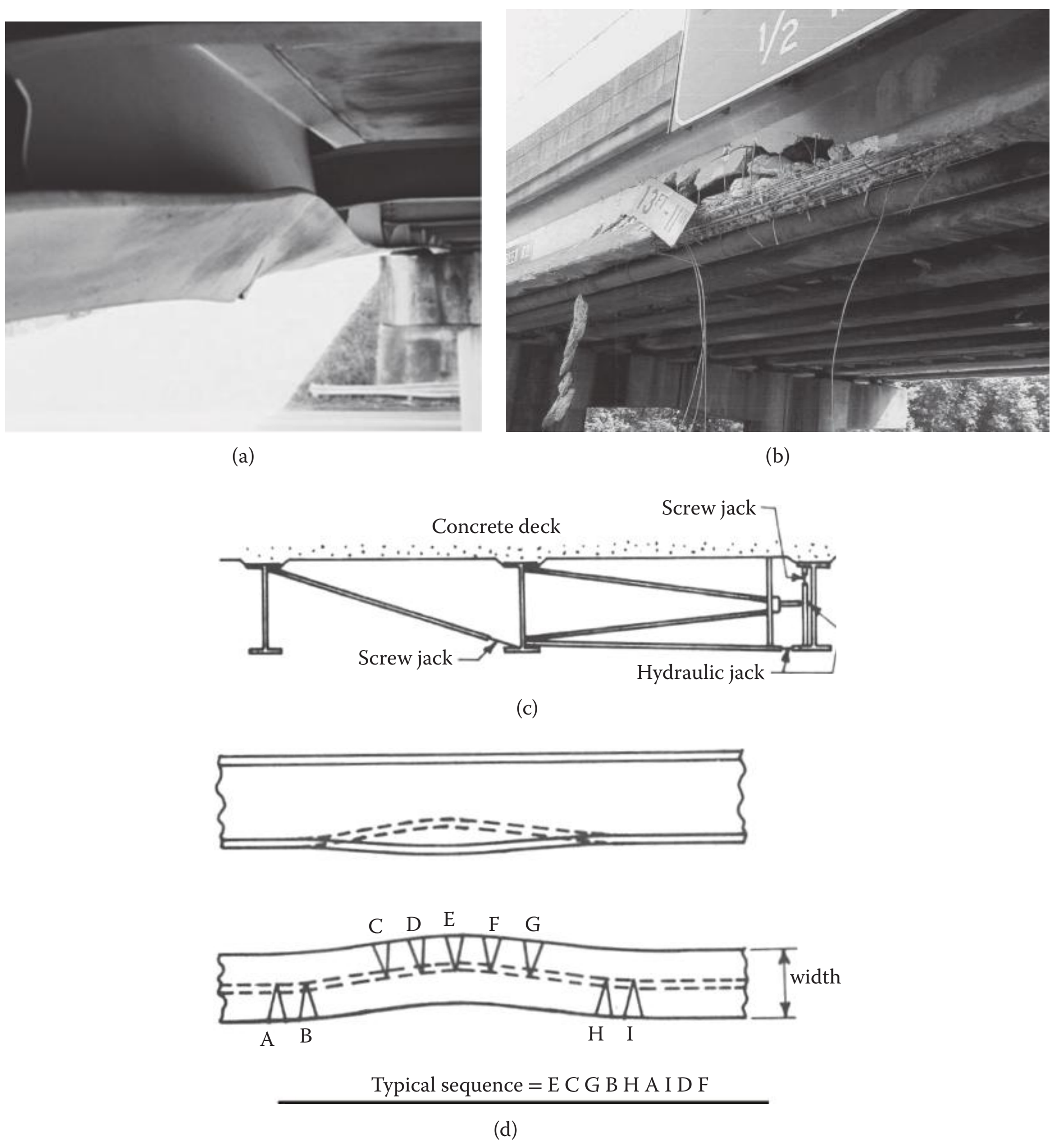


FIGURE 16.12 Typical vehicle impact damage and heat straightening repair: (a) steel girder, (b) concrete girder, (c) jacking system, (d) heating straightening sequence. (From NCHRP, *NCHRP Report 222*, Transportation Research Board, Washington DC, 1980.)

heat-strengthening basics, repairs, heat-strengthening effect on the material properties (strength, ductility, and toughness), and specifications for selection of contractors can be found in *FHWA Guide for Heat-Strengthening of Damaged Steel Bridge Members* (FHWA 2011b), as well as *NCHRP Report 271* (Shanafelt and Horn 1984).

16.4.6 Fatigue Damage Repair and Retrofit

16.4.6.1 Fatigue Damage

Fatigue damage is often discovered in old steel bridges with fatigue-prone details and subject to heavy truck traffic. There are two types of fatigue cases in steel bridges: (1) load-induced fatigue and (2) distortion-induced fatigue, as shown in Figure 16.13.

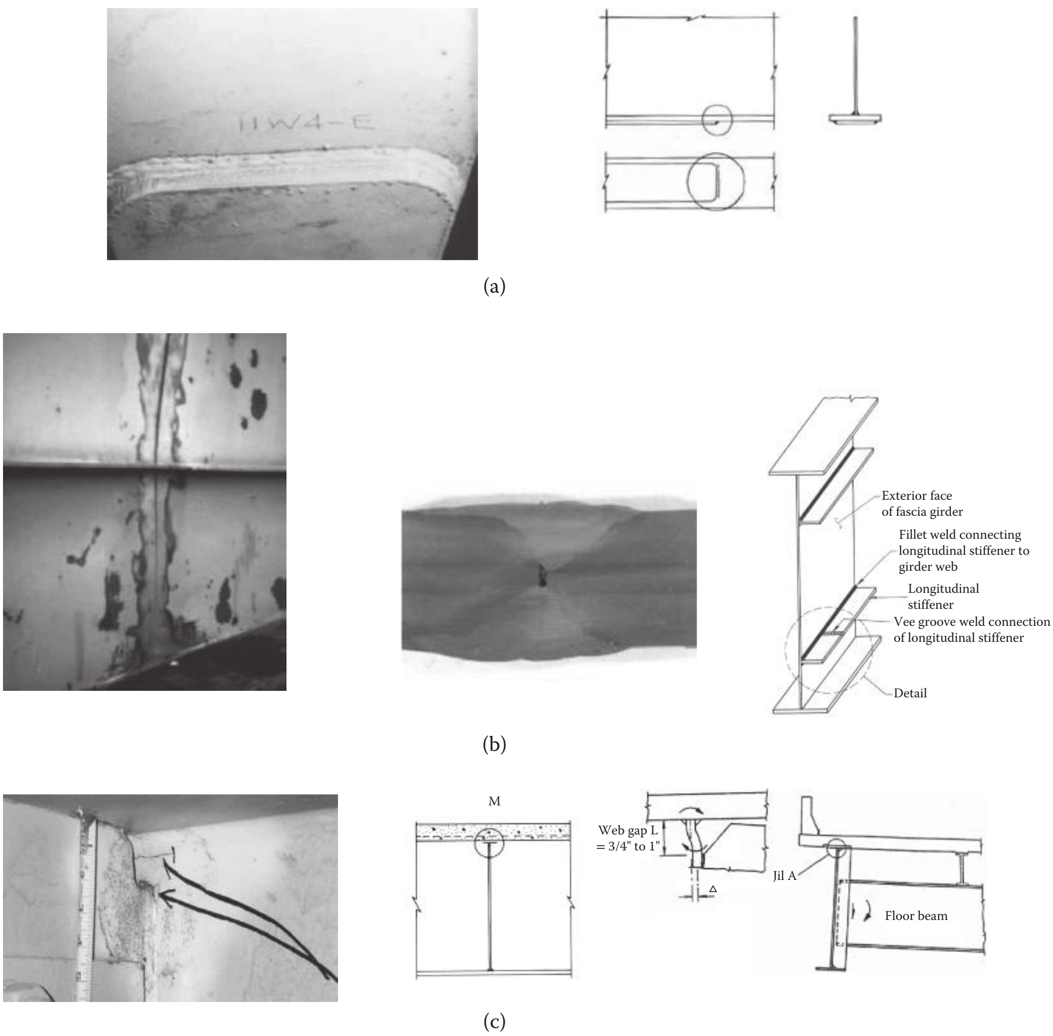


FIGURE 16.13 Typical fatigue cracks in steel bridges: (a) fatigue crack at cover plate detail (load-induced)–AASHTO Category E', (b) fatigue crack at groove welded longitudinal stiffener detail (load-induced)–lower than AASHTO Category C (lack of fusion weld without nondestructive testing and weld-reinforcement ground flush), (c) fatigue crack at diaphragm connection plate detail (secondary stress/distortion-induced).

"Load-induced fatigue" is fatigue damage attributed to directly repeated live load stress ranges subjected by a bridge detail. The accumulated fatigue damage depends on volume and weight (gross weight or axle weight) of heavy trucks traveling on the bridge, bridge configuration or load distribution, vibration and dynamic effects, the number of stress range cycles per truck passage, and detail category (geometry and weld defects) (AASHTO 2012a). Normally load-induced fatigue occurs in main members of the steel bridge because of relatively high stress ranges, or poor detail with large weld defects, or their combination. Load-induced fatigue is normally caused by global and in-plane stress ranges, and should be carefully controlled through the stress range in member detail design.

"Distortion-induced fatigue" is also called "secondary-stress-induced fatigue" and attributed to local relative distortion between bridge members or elements under repeated live load actions. Majority of fatigue damage in steel bridges in the United States is distortion-induced fatigue, which is more complicated and difficult to repair. Normally distortion-induced fatigue is caused by secondary stress ranges due to out-of-plane bending and the detail is redundant if fatigue cracks cut the member element. The problem of distortion-induced fatigue cracking has been reported in many types of steel bridges: two-girder and multi-girder bridges, trusses, and floor system of various steel bridges including suspension bridges, tied arches, and box girder bridges.

Typical distortion-induced fatigue occurs at the gap between diaphragm connection plate and girder top flange in negative moment region, that is, web-gap fatigue, as shown in Figure 16.13c. This is because the diaphragm connection plates were typically not welded to the top flange (tension flange), which was a requirement before the 1985 AASHTO bridge design specifications. Such a detail is no longer used in the new bridge design. The repair and retrofit methods are discussed in Sections 16.4.6.2 and 16.4.6.3.

In the United States, steel bridges built before the 1970s were not designed against fatigue. Although fatigue is not the main cause for deterioration of steel bridges compared with other types of deterioration, as discussed in Section 16.2.3.1, there have been many poor fatigue-prone details that caused fatigue problems in highway steel bridges (Fisher 1984). Fatigue issue has required extensive repair and retrofit work and is a big burden of maintenance. In a non-load-path-redundant steel bridge without proper inspection and maintenance, large fatigue cracks would put the bridge to a risk of bridge fracture, as discussed in Section 16.2.1. Particularly, distortion-induced fatigue has been a major issue and is difficult to repair. Currently, AASHTO requires fatigue design for new bridges (AASHTO 2012a), and fatigue inspection and evaluation for existing steel bridges, as part of regular inspection and FC inspection (AASHTO 2013). Fatigue design is of great importance in ensuring that steel bridge details have adequate fatigue life during bridge design life or infinite fatigue life.

16.4.6.2 Repair and Retrofit Methods for Fatigue Cracks

For existing bridges, the requirements for repair and retrofit will follow principles similar to new bridge design. When fatigue cracks are found by a routine or special inspection, several measures have to be taken depending on the type and extent of the cracking. Fatigue cracks maintenance and rehabilitation include the following:

1. Monitoring crack growth by regular inspection or fatigue sensors
2. Estimating crack propagation rate using fracture mechanics, or calculating residual fatigue life using Miner's rule
3. Drilling hole at crack tip to stop or retard crack growth
4. Using bushing to induce compression stresses around the stop hole drilled in front of the crack tip to suppress fatigue growth
5. Removing cracks by grinding and weld repair
6. Using post-weld treatment to introduce compression stresses at weld toe (Cheng et al. 2003) and enhance fatigue resistance, such as using air hammer peening or ultrasonic impact treatment
7. Strengthening/stiffening bridge members to reduce global stress level
8. Retrofitting details to reduce local distortion/stress level, and so on

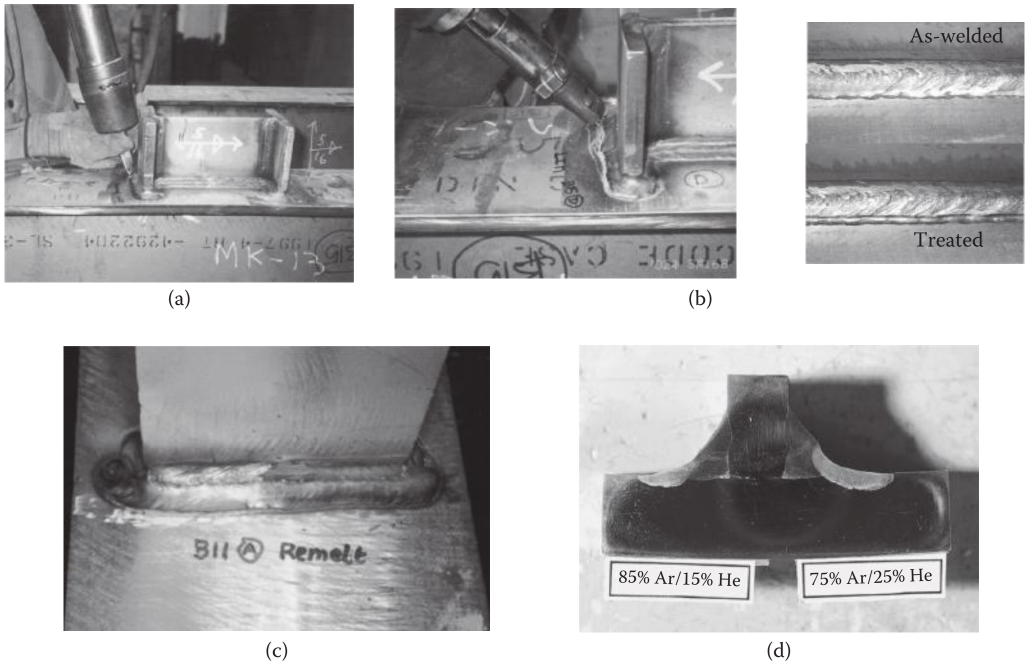


FIGURE 16.14 Methods of weld treatment and weld repair: (a) air hammer peening, (b) ultrasonic impact treatment, (c) gas tungsten arc remelting, (d) gas tungsten arc eliminated fatigue crack at weld toe.

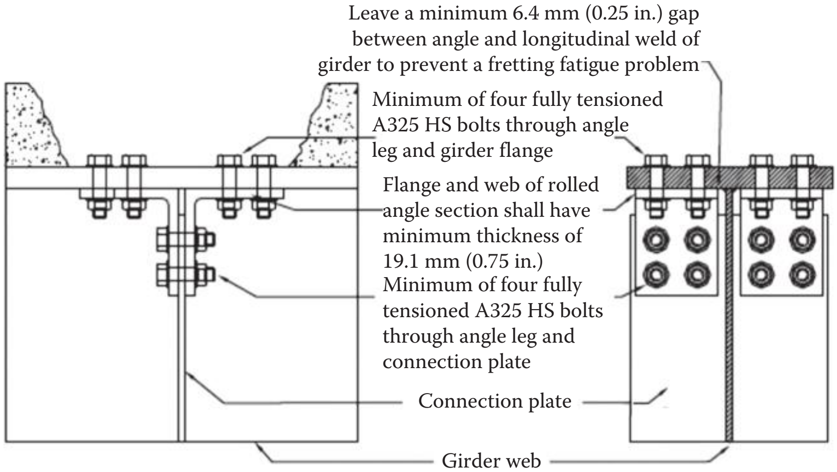
Figure 16.14 shows two post-weld treatment methods and one of weld repair methods (gas tungsten arc remelting) for very shallow fatigue cracking (Cheng et al. 2004a,b). Normally, post-weld treatment is intended to enhance fatigue resistance, whereas weld repair is used to restore the fatigue resistance.

16.4.6.3 Repair and Retrofit for Distortion-Induced Fatigue

Figure 16.15 shows two typical methods retrofitting out-of-plane bending-induced fatigue at the gap between the top girder flange and diaphragm connection plate, that is, "stiffening" and "softening." Stiffening method is to attach angles (by welding or bolting) to the top flange and connection plate so that the gap is stiffened to reduce the relative distortion and the out-of-plane bending under the truck traffic. This method is found to be effective to improve both local and system stiffness, but involving traffic disruption during rehabilitation work.

When the stiffening method results in a larger moment being transferred and a change of originally designed bridge behavior, the retrofit method has to be reevaluated, or softening method may be used. Softening method is to enlarge the gap to make the distortion more flexible to reduce out-of-plane bending stress ranges. This can be done by cutting down the connection plate not greater than 15 in., or drilling large diameter holes (3–4 in.) on the web. This method involves on-site drilling and grinding work, and stress relief should not be too great to change the general bridge behavior.

Figure 16.16 shows another example of distortion-induced fatigue in a 17-span steel girder bridge in New Jersey (Cheng et al. 2009). Fatigue cracks are all similar, that is, horizontal cracks in floor beam webs and vertical cracks in floor beam-to-girder bracket connection angles. The cracks are repaired by hole-drilling method and monitored in the following routine inspections. Over the years, the number and extent of the fatigue damage have continuously increased. Some cracks further propagated beyond the previously drilled holes. It appears that the fatigue damage was attributed to out-of-plane bending resulting from relative displacement between the concrete deck and the top flange of the main girder as live load passes across the bridge as well as differential temperature movement between the deck and



(a)

(b)

FIGURE 16.15 Typical retrofit methods for web-gap fatigue: (a) stiffening (one-side or two-side angle), (b) softening. (Reprint from Dexter, R. J., Ocel, J. M., *HWA-IF-13-020*, March, Federal Highway Administration, Mclean, VA, 2013. With permission.)

main girders. Two pilot studies were carried out by using electrochemical fatigue sensors to test fatigue crack propagation, and by using bushing in stop hole in front of the fatigue crack tip to suppress fatigue growth, as shown in Figure 16.16.

The urgency and priority of repair and retrofit of fatigue cracked details may be determined by the consequence of fatigue cracking. In many cases, existence of distortion-induced fatigue cracking may not have impact on the bridge structure load-carrying capacity because it normally occurs at the intersection of bridge members. However, if the fatigue cracks initiate and significantly propagate into a main girder element for distortion-induced fatigue, or load-induced fatigue such as cover plate detail, length of fatigue cracks will severely affect bridge safety. Particularly in a two-girder system bridge (noncomposite or composite), loss of one girder because of large fatigue crack may cause entire bridge failure; therefore this type of a girder is called FC member and special consideration and requirements are mandated for design, use of material, fabrication, inspection, and evaluation. Because material toughness, weld detail, weld quality, steel temperature, and loading speed are the most important factors for

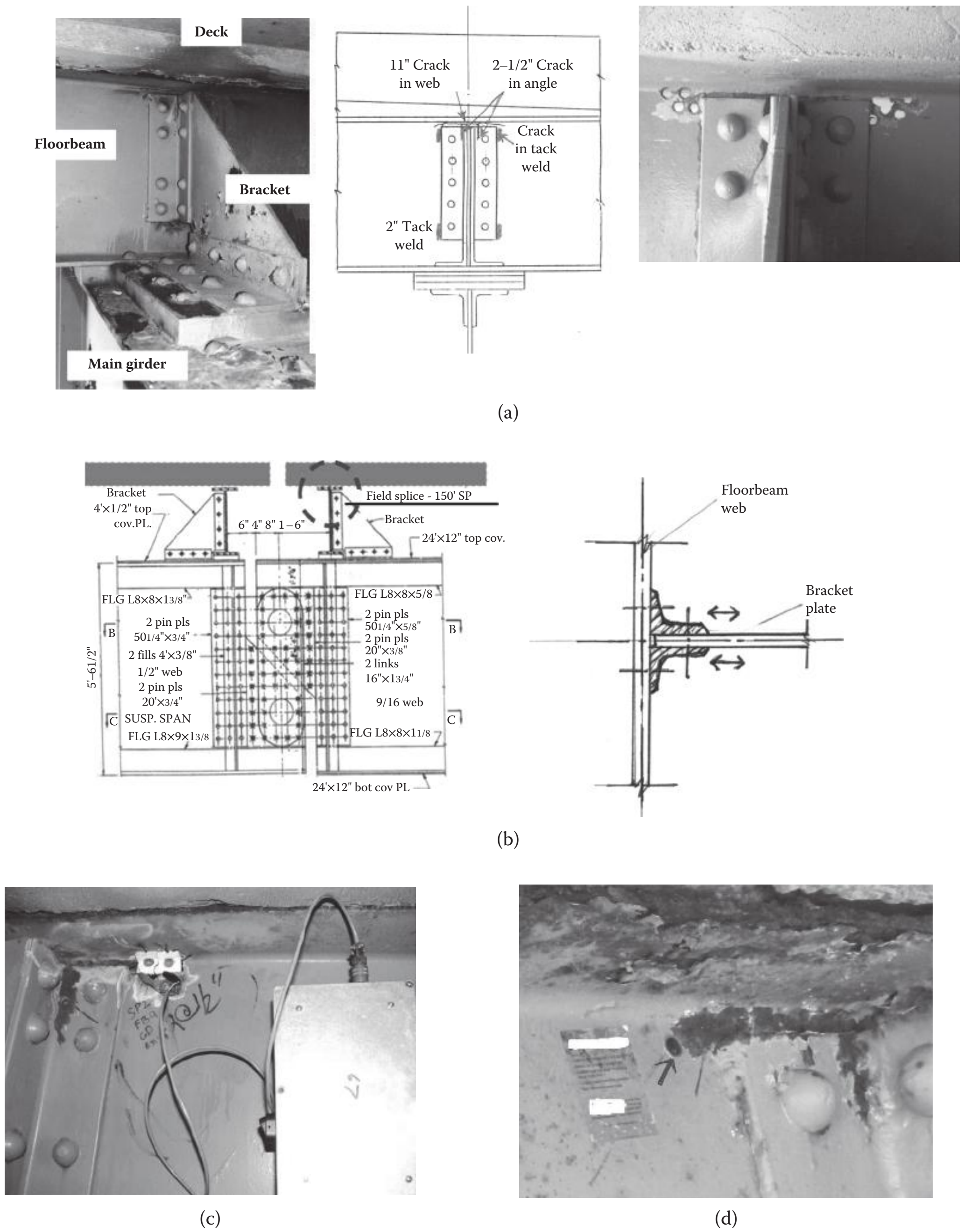


FIGURE 16.16 Fatigue damage, repair, monitoring, and suppressing for NJDOT Route 72 Manahawkin Bay bridge: (a) connection detail, fatigue cracks and hole-drilling repair; (b) relative movement between concrete deck and main girder caused out-of-plane bending; (c) fatigue crack growth monitoring using electrochemical fatigue sensor; (d) stop crack bushing.

FC control, it is hard for existing bridges to change these factors. Therefore, the most effective way to control FC members/bridges is to retrofit the fatigue-prone details and increase internal redundancy or system redundancy of the bridges.

16.4.7 Seismic Retrofit for Low Seismicity

Many bridges in the United States are vulnerable to seismic loads and could be seriously damaged or suffer collapse even in a relatively small earthquake because they have not been designed against seismic loads. The document (FHWA 2006) states that "…although the risk of bridge collapse is lower in the central and eastern United States, ground motions large enough to cause damage have a 10 percent chance of occurring within the next 50 years in 37 of the 50 states." To overcome this situation, retrofitting, strengthening, or replacing seismic-deficient bridges is urgent and necessary, especially for those bridges that are located along the highway of lifelines. Most seismic retrofit works involve bearings, superstructures, substructure (pier and bent), and foundations. Especially substructure and foundation retrofits are required by major seismic enhancement and thus involve intensive design and construction efforts. Because the topics on seismic design and seismic retrofit/strengthening are discussed in several chapters in *Seismic Design* of this Handbook, this chapter is limited to some superstructure retrofit for low seismicity that may be executed simultaneously along with the nonseismic retrofit and strengthening work.

16.4.7.1 Seat Length and Anchor Bolt Connections

Bearing systems to resist horizontal seismic movement may include bearings, anchor bolts, shear keys, bumper blocks, catcher blocks, keeper bars, and similar stoppers. Expansion rocker bearings (Figure 16.9) are most vulnerable to shear force from seismic loads of all types of bridge bearings because they usually are of a high aspect ratio, difficult to restrain and unstable, and can overturn after limited movement, especially for skewed bridges. Fixed rocker bearings with high aspect ratio are also very vulnerable to earthquake movement. The designer should recommend all existing rocker bearings (except suspended span and truss span) be replaced by isolation bearings. Elastomeric bearings (lead core elastomeric bearings) are normally stable during an earthquake although they have been found to fail due to inadequate strength or fastening under severe seismic shaking. If the condition of existing rocker bearings is not satisfactory even for nonseismic loads, such as being corroded and malfunctioning, the bearings have to be replaced anyway as part of rehabilitation program. In such a case, regular elastomeric bearings may be considered as an economical solution to improve seismic behavior of the bridge.

Large transverse or longitudinal movements may occur at bearings leading to loss of support and span collapse. The seismic movement/displacement depends on many factors and an equation is recommended as a basis for checking the minimum longitudinal support length, N (inch or millimeter) (FHWA 2006). When this minimum seat length is not satisfied, there is a risk of loss of support and span collapse. Several measures can be taken to solve the problem: (1) concrete seat extension, this is normally used when the abutment can be extended using the same foundation; and (2) bumper block or catcher block, when concrete extension may involve temporary formwork in addition to extended reinforcements, as shown in Figure 16.17.

Seismic loads are transferred from bearings to substructure through anchor bolts. The longitudinal and transverse loads can be evaluated by approximate methods or refined methods given in FHWA (2006). When the seismic capacity is not satisfied, additional anchor bolts or replacement of larger anchor bolts may be desired. The retrofit methods are similar to those described in Section 16.4.2.1.

16.4.7.2 Cable/Bar Restrainers

Some bridge collapse can result from the loss of the support of superstructure at bridge expansion bearing seats between bridge spans. To prevent bridge span from falling down, addition of a cost effective approach is to add longitudinal restrainers. This method is intended to limit the relative displacement at

(a) (b)

FIGURE 16.17 Seismic retrofit for insufficient seat length: (a) bumper block, (b) catcher block.

FIGURE 16.18 Seismic restrainer system.

expansion joints and thus, decrease the chance of loss of support at these locations. A restrainer system normally carries only tension forces and should be designed to be in elastic range to resist the maximum seismic loads. The system includes cables or bars, and end of the cables/bars (anchor plate, anchor nuts, etc.). Figure 16.18 shows a typical seismic restrainer device used in Japan. More details of design and information about restrainer system can be found elsewhere (Silano 1993; FHWA 2006).

16.5 Methods of Strengthening for Superstructures

Strengthening an existing bridge, by using different methods as briefly discussed in this section, can restore or increase live-load-carrying capacity of the bridge system. Methods described in Sections 16.5.1 through 16.5.7 generally involve a bridge system, that is, strengthening and stiffening the bridge through global performance, whereas methods in Sections 16.5.8 through 16.5.12 involve strengthening or stiffening bridge members/components. The strengthening methods discussed in this section may be applied to primary load-carrying members, floor system members, transverse members, splices/connections, and secondary members. Selection of the methods should be based on investigation and analysis of bridge conditions and requirements of current and future services.

16.5.1 Replacement

This is the most straightforward method for bridge strengthening. Replacement of superstructures means that a deteriorated superstructure is replaced and upgraded to a new superstructure designed with new requirements for load capacity, serviceability, fatigue, and other special requirements (such as extreme events like earthquake, blast, or collision). Replacement of superstructures is normally preferred because the rehabilitation cost is so high that it can result in the same order of a replacement cost, or constraints of constructability and/or material/labor force availability for bridge rehabilitation/widening make the rehabilitation so difficult. The design of superstructure replacement should consider the capacity of existing substructures and foundations, and thus, determine if they need to be strengthened at the same time. Replacement is normally used for relatively small-scale bridges with prefabricated systems for the accelerated bridge construction strategy. Superstructure replacement for larger-scale bridges is more complicated, such as building a new bridge that should consider entire bridge system. The *NCHRP Report 222* (NCHRP 1980) and the FHWA document (FHWA 2009b) provide several recommended replacement systems for bridge decks, prefabricated steel, timber, and concrete girder systems.

16.5.2 Additional Supports/Span Length Reduction

This method is often used as temporary strengthening when a bridge is in lack of load-carrying capacity during construction. The girder or member load-carrying capacity can be enhanced by making one span into two shorter continuous spans, thus reducing the moment and shear forces in the bridge members. If the space under a bridge allows additional piers and foundations, this method can be used as a permanent strengthening as well. This method is more used in building structures than in bridge structures because of the difficulty of new pier/foundation installation. When an additional support is installed, the supporting structure should be analyzed to adequately carry the new forces resulting from the additional supports.

16.5.3 Modification of Simple Spans (Continuity)

Many old steel bridges built consist of multiple simple spans because of construction simplicity. However, because of existence of many deck joints, corrosion resulting from joint leakage has become a big burden of bridge maintenance. Bridge bearings, floor system, and girders may be rusted and lose the load-carrying capacity. Such simple spans can be converted into a continuous span by connecting the adjacent spans and eliminating the joints. This modification reduces the magnitude of the positive moment at mid-span, whereas it increases the negative moment at the connected joints. The connection at the supports should be designed to resist both the increased moment and shear forces. Strong diaphragm system is normally provided at the location to increase shear capacity and global stiffness of the bridge system. Figure 16.19 shows an example of "simple span made continuous" steel plate girder detail using casted top steel bolted splice with bolted web plate (BEST 2000). The top flanges of steel girders are spliced with bolted plates and embedded in the composite concrete deck slab, whereas the girder webs are also spliced with bolted plates forming a strong connection with continuity.

16.5.4 Enhancement of Redundancy/Modification of Load Path of Bridge System

When a bridge system is non-load-path-redundant, failure of one member may result in partial or entire bridge failure, that is, the bridge fails to satisfy a certain level of load-carrying capacity by either collapse

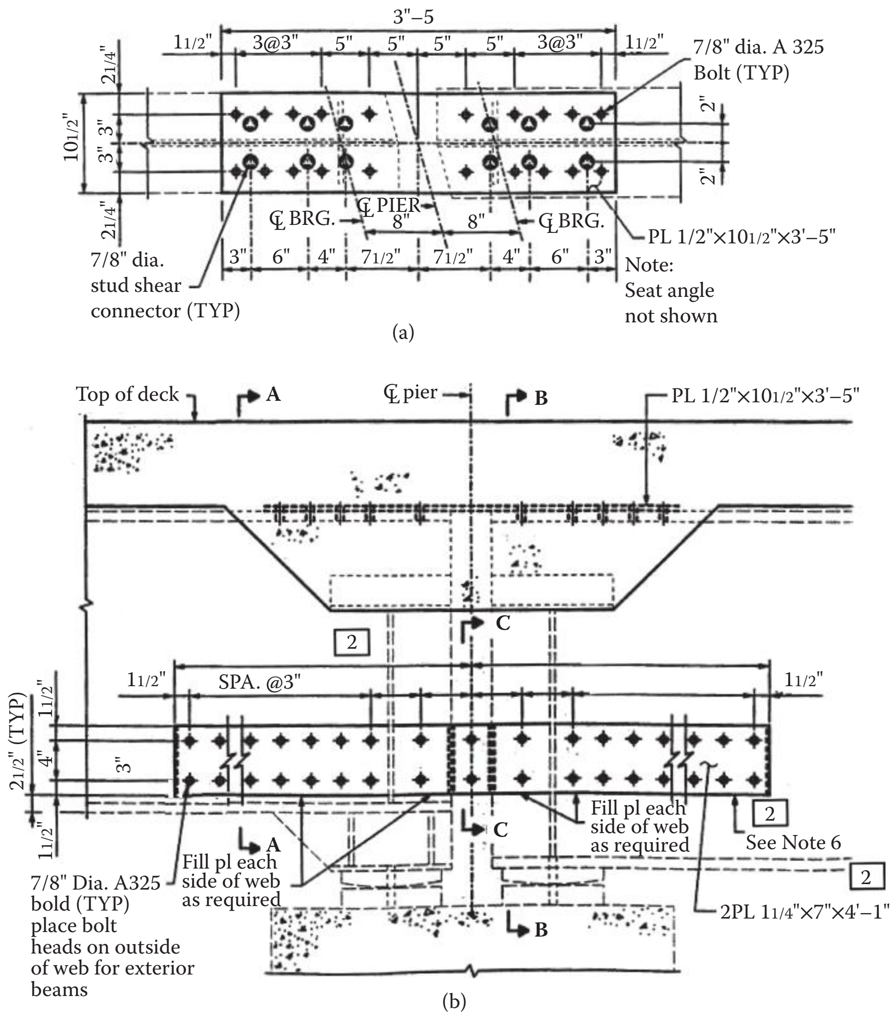


FIGURE 16.19 Example of continuity connection of steel plate girders: (a) plan, (b) elevation. (From BEST, *Study Report*, The Bridge Engineering Software and Technology (BEST) Center, University of Maryland, College Park, MD, 2000.)

or out of functioning because of overstress and/or excessive deflection, without alternative load paths. For steel bridges, such a bridge member that is in tension is classified as fracture critical member (FCM).

The NBIS (FHWA 2010a) defines an FCM as "a steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse."

The AASHTO *Manual for Bridge Evaluation* (AASHTO 2013) provides its definition as "fracture critical members or member components are steel tension members or steel components of members whose failure would be expected to result in collapse of the bridge."

The member corrosion or fatigue cracking may cause excessive loss of cross section, and consequently cause member fracture and eventually bridge system failure. Although corrosion or fatigue of steel members may be a slow process, member fracture can be a quick unstable brittle process if the steel material is not sufficiently tough, especially under cold temperature. The Silver Bridge collapse discussed in Section 16.2 is a classic example of bridge collapse caused by nonredundancy. In many cases, when a fatigue crack reaches a long size or its critical size, the member may fracture. This is the reason why the terms "fatigue" and "fracture" are often connected in steel bridge design. FC bridges need special care for design, fabrication, in-service inspection and maintenance, that is, fracture control plan, which demands significant additional costs. Therefore, a new bridge should be initially designed as a load-path-redundant system to ensure the bridge integrity and eliminate the labor-consuming and costly inspection/maintenance requirements (FHWA 2012b).

There are three classifications of redundancy: (1) internal redundancy, (2) structural redundancy, and (3) load-path redundancy.

Internal redundancy is a member configuration that contains three or more elements that are mechanically fastened (riveted or bolted) together so that multiple independent load paths are formed, that is, alternative and sufficient load paths exist within the member itself.

Structural redundancy is a configuration that provides continuity of load paths from span to span, that is, if member failure occurs, loading from that span can be safely redistributed to the adjacent spans and bridge failure may not occur. For example, in a two-span continuous girder the failure of a negative-moment region is not critical to the survival of a superstructure if the positive-moment region is sufficient to carry the load as a simply supported girder.

Load-path redundancy is a configuration that has three or more main load-carrying members, or an alternative and sufficient load path that exists between structural units. That is, if one member were to fail, the load would be safely distributed to other members, therefore bridge failure would not occur. If the additional redistributed load fails the alternative load path, progressive failure occurs, and the member could be FC. All primary and secondary members should be considered in determining the sufficiency of alternative load paths.

For an existing bridge that has lack of redundancy, there is a risk of bridge failure when the FCM fails. To avoid this happening, engineers may repair/retrofit the FCM and/or upgrade the bridge system to a redundant bridge by taking proper measures based on the knowledge of the existing bridge configuration, current and target load paths/redundancy, and target load-carrying capacity of the FCM or the bridge system. Although the currently available AASHTO documents do not provide information of quantitative degrees of redundancy, an engineer can use refined analytical methods to analyze and prove load paths and redundancy of a bridge by simulating fracture of FCM and any bridge members. The strengthening methods for the FCM can refer to the method described in this section. For FCM with fatigue cracks, the fatigue cracks should be timely monitored and repaired, or the member should be strengthened. The improvement of system redundancy may depend on the bridge configuration and existing damage using the three types of redundancy to reach cost-effectiveness.

16.5.5 Application of Prestressing with External Posttensioning Cables/Rods

This method is a very effective means to increase load-carrying capacity of a bridge although it is somewhat expensive and elaborate (Daly and Witarnawan 1997). The posttensioning members can be composed of high-strength steel cables or rods. Care should be taken in designing cable/rod stresses and detailing the anchorages to transfer the tensile forces to the beams. Figure 16.20 shows a sketch of this technology, including (1) vertical support (i.e., "king post" support) at one or two positions along the beam, and (2) prestressing the lower flange (into compression) with cables/rods to the desired level of tension stresses and with a small vertical force at the support. Consideration must be given to local stresses in beams induced by the intermediate support. Corrosion protection should be provided to the cables/rods and the anchors. The prestressing concept can also be used to reduce tension stress in negative moment region by applying prestressing cables near the top flanges. Prestressing cables can be either steel cables or carbon fiber composite cables that have been developed for more than 20 years although there have been much more steel cable applications.

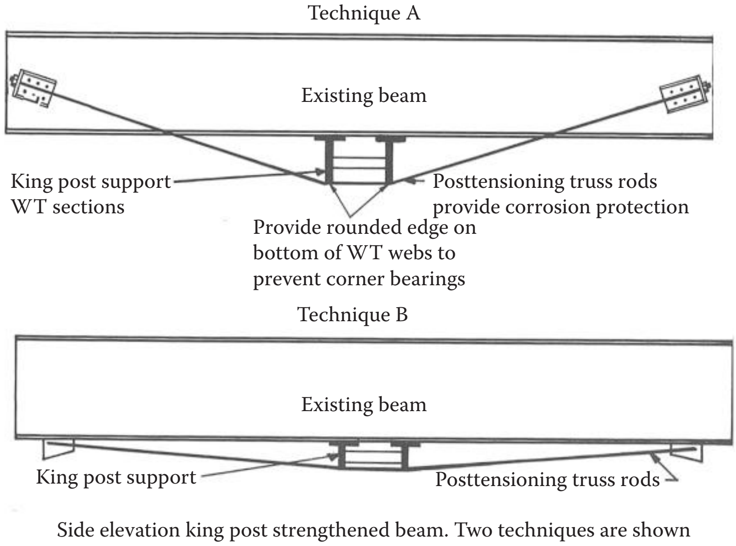


FIGURE 16.20 Steel girder strengthening by external posttensioning prestressing.

Figure 16.21 shows typical posttensioning details for concrete girders (DELDOT 2005) where the symmetrical jacking corbels are placed on either side of the damaged area, in the sound sections of the beam, and anchoring them to the bottom flange. Posttensioning tendons are passed through the corbels and anchored against the bearing plates.

Figure 16.22 shows California Island View Sidehill Viaduct strengthened by external prestressing. The Island View Sidehill Viaduct, carrying two-lane highway traffic, with four spans of 26.4, 27.1, 27.1 and 27.3 m, and width of 12.5 m, was built in 1968. The superstructure consists of five simply supported precast prestressed concrete I-girders composited with concrete deck. Abutment 1 and Bents 2 are founded on steel piles. Bent 3 and Abutment 5 are founded on spread footings.

To increase live load-carrying capacity, the 3 in. asphalt concrete overlay and membrane deck seal were removed and replaced with 1.5 in. thick overlay of LMC. All precast girders were strengthened by the externally applied prestressing strands in 1986 (Figure 16.22). A recent inspection discovered that numerous nuts connecting the metal brackets to the grouted 0.375 diameter bolts in the prestressed concrete girders have loosened and/or are missing. It is recommended to tighten those loose nuts and replace any missing nuts and washers throughout the structure to prevent any future fatigue damage to the girder that may be caused by movement of those rods and connecting brackets.

16.5.6 Composite Action

Although it is well known today that composite action can increase the moment of inertia of bridge beams/girders by using stud shear connectors between concrete deck slab and the top flange of beam/girder, many steel bridges built before the 1960s are noncomposite because of the lack of knowledge of shear connector application (welded channel, welded stud, and other types of shear connectors) at that time. Because composite action can be added to an existing bridge to increase bridge strength, this method can be used when a noncomposite deteriorated concrete deck is to be replaced by a new deck, or a noncomposite superstructure is to be replaced. If a deck is still sound, composite action can be added by simply drilling holes through the deck, adding shear connectors on the girder top flanges and grouting the holes (Figure 16.23). It is very important that the composite action should be designed to meet the requirement of full composite bridge (shear connection strength, embedment height, space, etc.) so that the composite action can be accounted for strengthening the bridge (AASHTO 2012b).

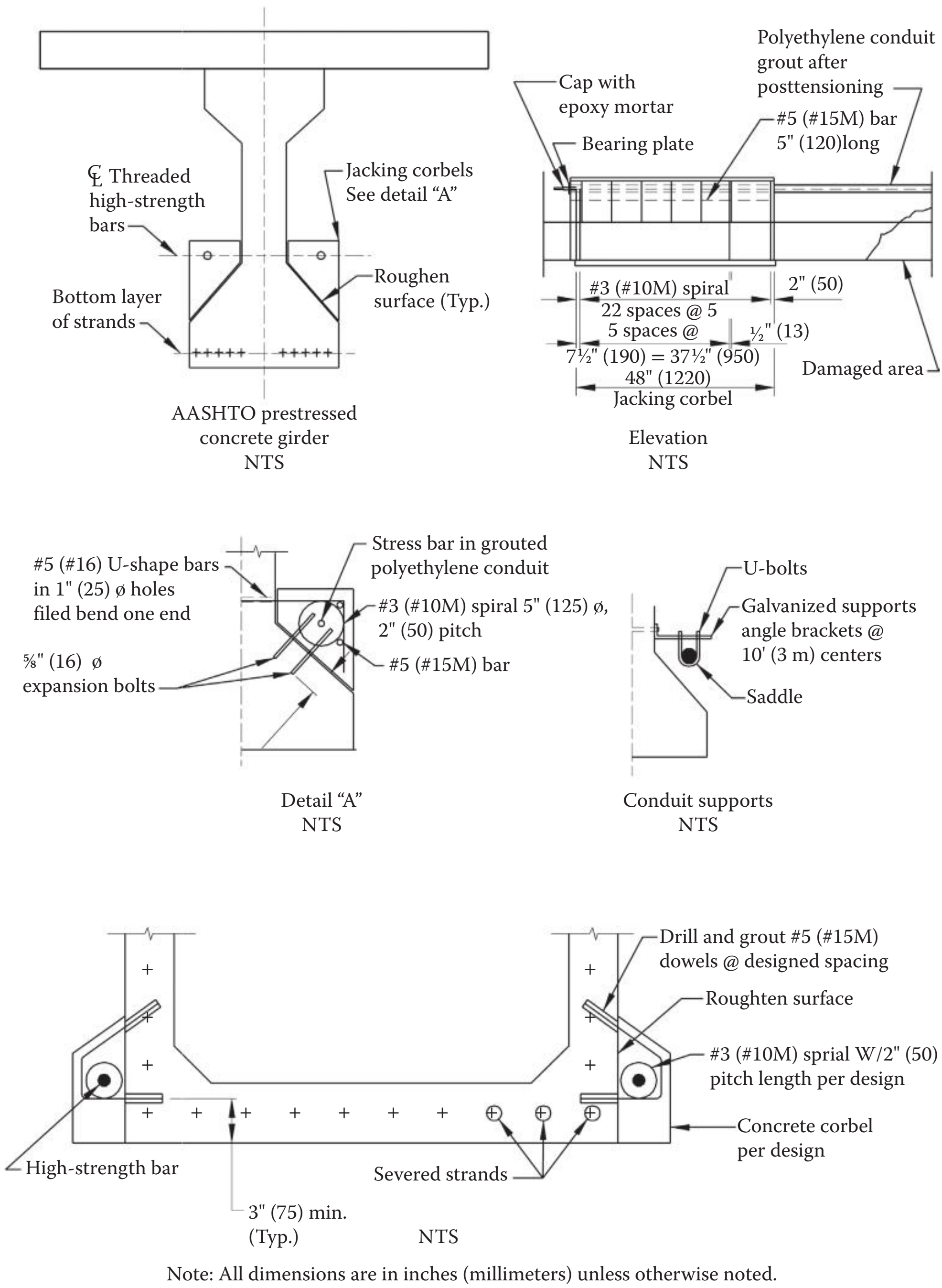


FIGURE 16.21 Typical posttensioning details for concrete girders. (From DELDOT, *Bridge Design Manual*, Dover, DE, 2005.)

(a)

(b)

FIGURE 16.22 Island View Sidehill Viaduct strengthened by external prestressing: (a) girder bottom view, (b) girder side view.

16.5.7 Additional Members/Additional Transverse Bracing to Bridge Structures

This method is aimed to increase load-carrying capacity of a bridge system through three-dimensional load distribution rather than strengthening individual members. Adding standard steel shapes or transverse bracing may be a cost-effective way to increase system stiffness, improve load distribution behavior, and thus reduce stress and distortion of individual members. Another option is to install new steel beams between the existing beams as long as the bridge can accommodate the increased dead load.

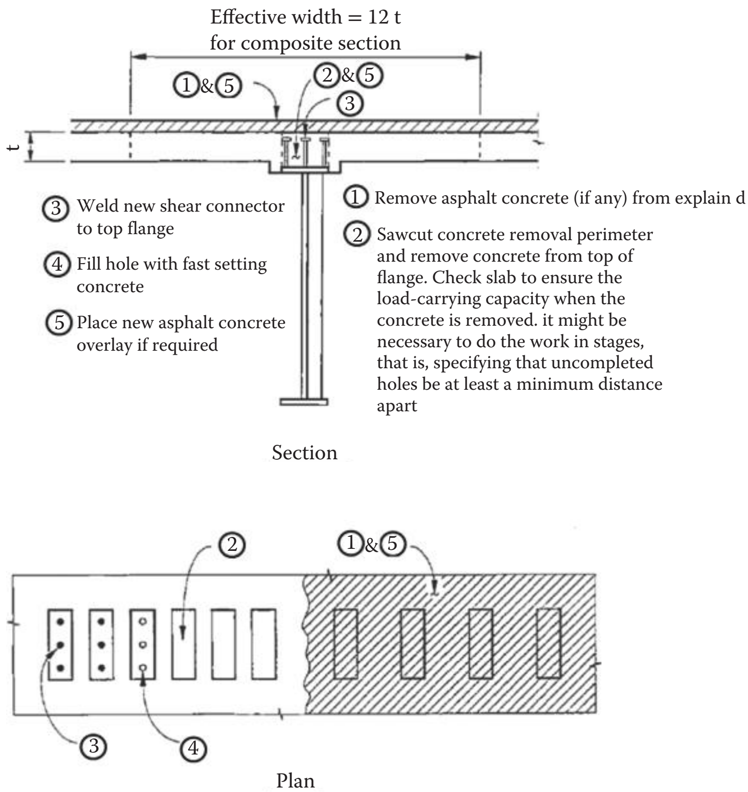


FIGURE 16.23 Providing composition action (Courtesy of California Department of Transportation).

16.5.8 Enhancement of Member Cross Section (Doubler Plate, Cover Plate, or Stiffeners)

This is a straightforward method in theory and practice. Steel cover plates are normally used in a plate girder flange to increase the section modulus (thus flexural capacity), and used in truss bridge members to increase the cross-section area and reduce slenderness ratio. Doubler plates are generally used to increase shearing capacity or buckling resistance of the deteriorated girder web, by welding or bolting doubler plates to the web. This method is also used to repair vehicle collision damage, as shown in Figure 16.24. Adding stiffeners may increase local buckling resistance of steel girders and shear strength at the bearing support location.

Figure 16.25 shows metal sleeve splice strengthening for a damaged prestressed concrete beam (DELDOT 2005). It does not normally restore prestresses, although partial or full prestresses may be restored by preloading.

Although this approach appears simple, sometimes in situ condition, (e.g., the traffic under the bridge needs to be maintained all the time), it does not allow the simple strengthening. In this case, alternative strengthening approach has to be considered. Also, it should be noted that if a cover plate or stiffener is welded to a load-carrying member for strengthening, fatigue resistance especially for the welded cover plate detail should be checked because this detail is very prone to fatigue with AASHTO Fatigue Category E′ (AASHTO 2012a). Some owners prohibit such a detail or require special approval for its use (NJDOT 2009). If these plates/stiffeners are welded to bridge members, fatigue resistance has to be checked.

(a) (b)

FIGURE 16.24 Repair and strengthening of steel girder web damage by vehicle impact: (a) damaged girder, (b) repaired girder.

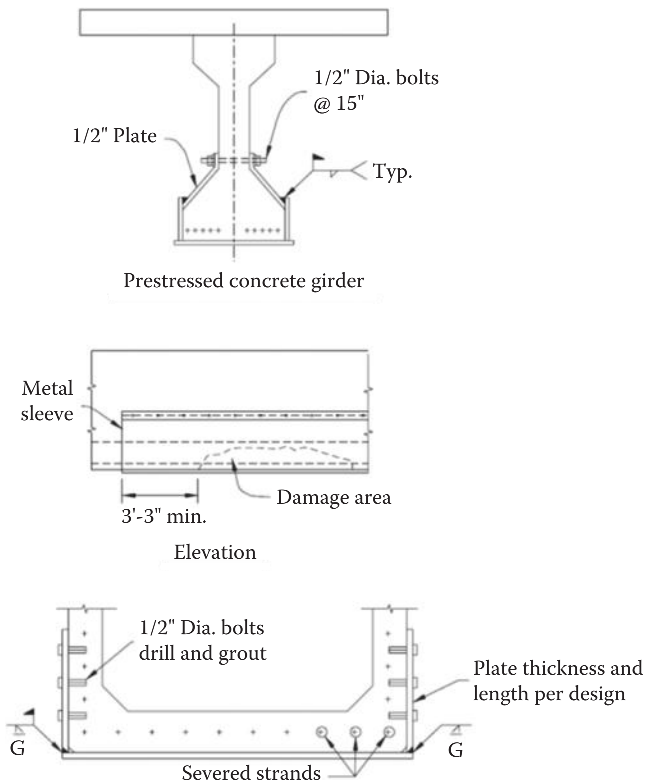


FIGURE 16.25 Metal sleeve splice strengthening of concrete girders.

16.5.9 Dead Load Reduction

This is a very straightforward method to increase load-carrying capacity of a bridge structure. There are many methods to reduce dead loads when bridge rehabilitation is carried out, such as using lightweight decks to replace existing decks (as discussed in Section 16.4.1), using lightweight concrete for concrete work, using steel girders to replace concrete girders, using thinner overlay, using FRP for girder or pier repair/retrofit, or any methods such that the net weight change is reduced after rehabilitation work is completed.

16.5.10 Concrete Encasement

This method has been used in many projects in the past because of its simplicity for strengthening load-carrying capacity for bending and buckling of steel girders, particularly when other methods encountered difficulties in welding additional stiffeners/plates in the field, or when working in a confined space. This method is especially suitable when used directly over the supports where the additional weight of concrete is less critical and forms are easy to be installed. However, because of large increase of the dead load, the use of this approach is limited. In addition, the past experience has shown that cracking in concrete encasement trapped moisture and water into the strengthened girders, resulting in cross-sectional loss of steel girders. Therefore, this method has to be carefully used to ensure that there is no major concrete cracking that would cause a corrosion issue.

16.5.11 Fiber-Reinforced Polymer Bonding/Wrapping or Steel Plate Bonding

To avoid disadvantage of heavy concrete encasement, FRP (with 1/5 concrete weight) sheet wrapping with epoxy adhesives is one of the new technologies to strengthen girder shearing and flexural capacities. Debonding at the end of the carbon fiber sheets/plates needs to be controlled by mechanical fixings. Because the elastic modulus of the carbon fiber is lower, it may restrict the strengthening range when considering the two materials working together.

In this case, epoxy adhesives used to attach steel cover plates may be another option to strengthen bridge girders. The adhesive material is now developed to be sufficiently strong to make steel plate bond to an existing girder. Bolting down the ends of plates is required to avoid end pull off. In addition, a recent study has been carried out for strengthening welded connections under fatigue loading using carbon FRP and the results showed that attaching carbon FRP doubler elements is a viable technique and can lead to reduction in stress demand at fatigue-critical welded connections (Kaan et al. 2008; Rolfe et al. 2013). However, the application to real structures is still limited due to limited data.

AASHTO recently published the first edition *Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements* (AASHTO 2012b). It provides design details in materials, surface preparation, loads and load combinations, and design for members under flexure, shear and torsion, and combined axial force and flexure.

Figure 16.26 shows the installation of a glass FRP fabric to strengthen deteriorated reinforced concrete T-Beam.

FIGURE 16.26 Glass fiber-reinforced polymer fabric to strengthen deteriorated reinforced concrete T-beam. (Courtesy of FHWA.)

FIGURE 16.27 Severe pack rust in riveted connections of steel bridge.

16.5.12 Enhancement of Member Splices/Connections

16.5.12.1 Strengthening Connections or Replacing Rivets by High-Strength Bolts at Connections/Splices

Bridge member splices/connections are normally considered to be overmatched compared to the strength of member themselves. Many old bridges used riveted connections and have been found to be deteriorated, such as loss of rivets, bending of rivets, bearing failure of the holes, and severely corroded with pack rust. Figure 16.27 shows such a riveted connection with severe pack rust in Route 72 Manahawkin Bay Bridge (Cheng et al. 2009). Nowadays new types of high-strength bolt connections are designed to resist slipping between the splice/connection plates and the bridge elements, and installed by using a torque wrench or direct tension indicator to induce high pretension in the bolts so that all plates at the splice/connection are tightly connected so that loss of bolts, bending of bolts, and corrosion are not formed as riveted connections.

16.5.12.2 Gusset Plate Connection of Truss Bridges

FHWA has published *Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges* (FHWA 2009a). A newly published *NCHRP Report 197* (Ocel 2013) presents the findings of an experimental and analytical investigation exploring the failure modes of gusset plate connections in steel truss bridge based on a recommendation provided by the NTSB at the conclusion of the I-35W Bridge collapse investigation. Primarily the research focused on buckling and shear failure modes of gusset plates, including the effects of section loss, multilayered plates, and edge stiffening. Design and rating resistance equations with calibrated factors developed according to a load and resistance factor philosophy are ready for adoption into the AASHTO *LRFD Bridge Design Specifications* (AASHTO 2012a) and the AASHTO *Manual for Bridge Evaluation* (AASHTO 2013).

16.5.13 Combination of Two or More Methods

Apparently, in the field one method or combined methods can be flexibly selected by the engineer on project-by-project basis. It is noted that although the methods discussed earlier have often been used in structural strengthening, this section does not cover all the strengthening methods. Depending on uniqueness of each individual project, other methods may be used, and new technologies are still being developed.

16.5.14 Steel Bridge Repair Example

Figure 16.28 shows a cracked and corroded cap beam and stringer. The web of the cap beam near bearing pedestal of the stringer contains two wide corrosion holes (4 in. high × 1 in. wide) connected by a vertical crack of 2 in. long. A vertical crack of 0.375 in. long emanates from bottom of lower hole. Below the bearing pedestal, there is a corroded area with up to 45% section loss. Localized about 30% overall section loss exists in high shear area of the cap beam web around the pedestal.

Figure 16.29 shows the repaired cap beam and stringer. This was an urgent repair to reinforce the cap beam web and maintain stringer support. The repair procedure included the following steps: installing

FIGURE 16.28 Cracked and corroded cap beam and stringer.

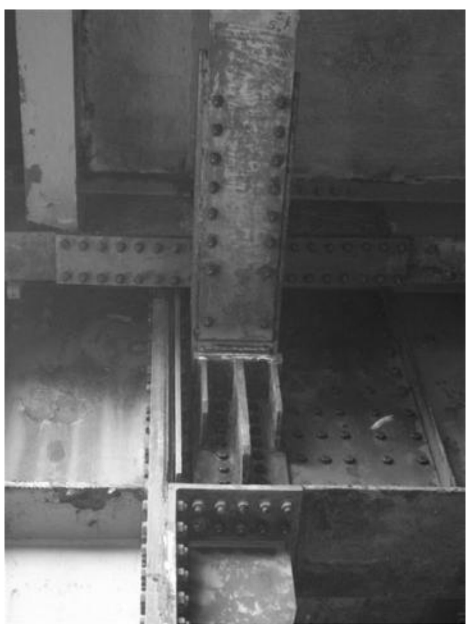


FIGURE 16.29 Repaired cap beam and stringer.

temporary supports for the shown stringer (Figure 16.28) at this expansion side of the cap beam and for a stringer at the other (fixed) side; installing a new bracket connecting the cap beam to the column; removing guide angles at shown stringer support and connection angles at fixed stringer support and replacing them with new angles; removing bearing seat; cutting and removing web corrosion holes and cracks and installing new plates to fill removed web areas; installing reinforcing plate for cap beam web (one on each side of cap beam); reinstalling bearing seat; and removing temporary supports. The new bracket was needed only as temporary support for the cap beam, but it was kept part of the permanent repair to provide additional structural redundancy. Other steps and details of this repair procedure are beyond the scope of this section.

16.5.15 Survey Questionnaire on Steel Bridge Repairs

A survey questionnaire was prepared in 2010 by Task Group 14 Field Repair and Retrofit of Existing Steel Bridges of the AASHTO/NSBA (National Steel Bridge Alliance) Steel Bridge Collaboration and sent to all state transportation agencies, as well as other agencies and firms in the United States. The intent of the survey was to help the group in developing a practical guideline document on typical repairs of steel bridges.

The survey listed 12 bridge or bridge element conditions including fatigue, fracture, corrosion, rivet/bolt deterioration, weld deterioration, low load capacity, nonredundancy, impact damage, fire damage, bearings deterioration, gusset plate deterioration, and eyebar deterioration. In addition, a condition category "other" was provided as an option for survey participants to define. A list of five questions, to be answered in relation to each specified condition of a bridge or a bridge element, was presented. The questions were as follows:

A. Have you experienced this condition?
B. Have you taken remedial action including monitoring?
C. Do you have repair/retrofit details?
D. Will you share the repair/retrofit details?
E. Do you want the document to include typical details?

The requested response for each question was a check mark indicating "Yes" or "No."

Participants who have experienced a specific condition (Question A) were requested to provide its frequency of occurrence by selecting one of three descriptive words: frequently, normally, or rarely. And if remedial action was taken (Question B) for that condition, participants were requested to estimate the percentage of its occurrence for each of three action categories: emergency repair, scheduled repair, and monitoring. These follow-up questions (or requests) were as follows:

A2. If you have experienced this condition, check its frequency of occurrence.
B2. If you have taken remedial action, estimate its percentage of occurrence.

Survey responses were submitted by 15 state transportation agencies (DOTs). A complete report on the survey data is under development. An abbreviated summary of preliminary results is shown in Table 16.1. Only data for questions A and B and follow-up questions A2 and B2 are listed. All values in the table are percentages, representing either the number of respondents who have experienced a specific condition (A and B), the frequency of occurrence of a condition (A2), or the frequency of occurrence of a remedial action taken (B2). For questions A and B, the responses were converted to 100% (for Yes) or 0% (for No). An empty response was assumed 0% occurrence for all questions. For follow-up question A2, the three descriptive words were assigned numeric values as follows: 90% for frequently, 50% for normally, and 10% for rarely. Values for question B2 are averages of actual percentages of occurrence provided by the respondents.

TABLE 16.1 Survey Questionnaire—Abbreviated Summary of Preliminary Results

					B2(%)		
	Bridge Condition	A(%)	B(%)	A2(%)	ER	SR	M
1	Fatigue	93	93	22	17	35	28
	Fracture	67	53	7	39	7	14
2	Corrosion	100	100	63	11	37	49
3	Rivet/bolt deterioration	80	73	30	6	15	59
	Weld deterioration	67	67	17	2	12	66
4	Low load capacity	93	93	20	0	21	66
5	Nonredundancy	87	67	25	1	3	55
6	Impact damage	100	93	34	29	26	31
7	Fire damage	67	67	11	34	13	6
8	Bearings deterioration	93	87	41	6	34	47
9	Gusset plate deterioration	80	73	17	1	13	59
	Eyebar deterioration	73	73	7	7	11	42

A. Have you experienced this condition?
B. Have you taken remedial action including monitoring?
A2. If you have experienced this condition, check its frequency of occurrence.
B2. If you have taken remedial action, estimate its percentage of occurrence.
ER, emergency repair; SR, scheduled repair; M, Monitoring

The following preliminary conclusions can be drawn from the survey data:

- Corrosion and impact damage are experienced by all respondents (100%).
- Fatigue, low load capacity, and bearing deterioration are experienced by most respondents (93%), followed by nonredundancy (87%).
- Fracture, weld deterioration, and fire damage are experienced by the least number of respondents (67%).
- Remedial action (question B) responses follow closely the responses to question A, with one exception. For nonredundancy, out of 87% respondents who experienced this condition, 67% of them took remedial action.
- Corrosion is distinctly at the top of the list in terms of its frequency of occurrence (63%), followed by bearings deterioration (41%).
- Fracture and eyebar deterioration have the least frequency of occurrence (7% for each).
- Emergency repair is performed mostly on fracture (39%), followed by fire damage (34%) and impact damage (29%).
- Scheduled repair is performed mostly on corrosion (37%), followed by fatigue (35%), bearings deterioration (34%), and impact damage (26%).
- Monitoring is performed mostly on weld deterioration and low load capacity (66% for each), followed by rivet/bolt deterioration and gusset plate deterioration (59% for each), nonredundancy (55%), corrosion (49%), and bearings deterioration (47%).

16.6 Summary

This chapter briefly discusses the various methods of rehabilitation and strengthening of existing highway bridge superstructures. Information is given on lessons learned from major highway bridge collapses, causes of bridge deterioration, decision-making considerations of rehabilitation and strengthening, as well as various methods of nonseismic-related rehabilitation and strengthening. Several case histories of strengthening are presented. For more detailed and additional information on bridge strengthening and rehabilitation, the reader may refer to Reid et al. (2001), Shanafelt and Horn (1984), Silano (1993), Xanthakos (1995), Dorton and Reel (1997), Khan (2010), Kim (2011), and Newman (2012).

References

AASHTO. 1976. *Interim Specifications Bridges*, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2012a. *AASHTO LRFD Bridge Design Specifications*, U.S. Customary Units, 2012, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2012b. *Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements*, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2013. *Manual for Bridge Evaluation*, 2nd Edition, with 2011 and 2013 Interim Revisions, American Association of State Highway and Transportation Officials, Washington, DC.

ACI. 2003. "Field guide to concrete repair application procedures–crack repair by gravity feed with resin," *ACI RAP Bulletin 2*, ACI Committee E706, American Concrete Institute, Farmington Hills, MI.

Alsup, R. 2006. "Bridging the gap: how to prevent disasters such as the collapse of the Lowe's Motor Speedway pedestrian bridge," *Final Report*, Prepared for Senator Elizabeth Dole, Office of Senator Elizabeth Dole, Washington, DC.

Bazant, Z. P., Yu, Q., and Li, G. H. 2012a. "Excessive long-time deflections of prestressed box girders. I: record-span bridge in Palau and other paradigms," *Journal of Structural Engineering*, 138(6), 676–686.

Bazant, Z. P., Yu, Q., and Li, G. H. 2012b. "Excessive long-time deflections of prestressed box girders. II: numerical analysis and lessons learned," *Journal of Structural Engineering*, 138(6), 687–696.

Bazant, Z. P., Yu, Q. et al. 2010. "Excessive deflections of record-span prestressed box girder–lessons learned from the collapse of the Koror–Babeldaob Bridge in Palau," *ACI Concrete International*, 32(6), 44–52.

Bazant, Z. P., Yu, Q. et al. 2011. "Excessive multidecade deflections of prestressed concrete bridges: how to avoid them and how to exploit their monitoring to improve creep prediction model," *FIB Symposium*, Prague, Czech Republic.

BEST. 2000. "Proposed design of existing simple span steel girders made continuous," *Study Report*, The Bridge Engineering Software and Technology (BEST) Center, University of Maryland, College Park, MD.

Burgoyne, C. and Scantlebury, R. 2006. "Why did Palau Bridge collapse?" *Brunel Bicentenary Conference*, July 607, Bristol Temple Meads, UK.

Burgoyne, C. and Scantlebury, R. 2008. "Lessons learned from the bridge collapse in Palau," *Proceedings of the Institution of Civil Engineers–CIVIL ENGINEERING*, 106(CE6), 28–34.

Cheng, X., Fisher, J. W., Prask, H. J., Gnaeupel-Herold, T., Yen, B. T., and Roy, S. 2003. "Residual stress modification by post weld treatment and its beneficial effect on fatigue strength of welded structures," *International Journal of Fatigue*, 25(9–11), 1259–1269.

Cheng, X. H. and Fisher, J. W. 2006. "Fatigue and fracture of steel bridges–design, research and practice," *Proceedings of 4th US–China Bridge Engineering Workshop*, June 19–22, Chongqing, China.

Cheng, X. H., Fisher, J. W., Yen, B. T., and Roy, S. 2004a. "Fatigue resistance of repaired and surface-treated AL-6XN stainless steel welds," *ATLSS Report No. 04-01*, January, Lehigh University, Bethlehem, PA.

Cheng, X. H., Fisher, J. W., and Yen, B. T. 2004b. "Fatigue crack inspection, weld repair and postrepair fatigue behavior of stainless steel welded components," *Proceedings of IABSE Symposium*, Shanghai, China.

Cheng, X. H., Renman, G. T., and Dunne, R. W. 2009. "Fatigue damage in the seventeen-span Rt.72 Manahawkin Bay Bridge," *Proceedings of International Conference on Fatigue and Fracture in the Infrastructure–Bridges and Structures of the 21st Century*, July, Philadelphia, PA.

CNN. 2000. Cause of collapse revealed, May 25, 2000, *CNN/Sports Illustrated*, retrieved January 18, 2013, from http://images.si.com/motorsports/2000/cocacola600/news/2000/05/25/collapse_report_ap/.

Daly, A. F. and Witarnawan, W. 1997. "Strengthening of bridges using external posttensioning," Road Research Development Project Published Paper PA 11, *TRL PA3307/07*, Transportation Research Laboratory, UK, in association with PT Yodya Karya, Indonesia.

DELDOT. 2005. *Bridge Design Manual*, Chapter 9–Rehabilitation of Existing Bridges, Delaware Department of Transportation, Dover, DE.

Dexter, R. J. and Ocel, J. M. 2013. "Manual for repair and retrofit of fatigue cracks in steel bridges," *HWA-IF-13-020*, March, Federal Highway Administration, Mclean, VA.

Dorton, R.A. and Reel, R. 1997. "Methods for increasing live load capacity of existing highway bridges," *NCHRP Synthesis of Highway Practice 249*, National Cooperative Highway Research Program, Transportation Research Board, Washington, DC.

FHWA. 1970. "Collapse of U.S. 35 Highway Bridge, Point Pleasant, West Virginia, December 15, 1967," *Highway Accident Report*, December, Federal Highway Administration, Washington, DC.

FHWA. 1984. "Collapse of a suspended span of Route 95 Highway Bridge over the Mianus River, Greenwich, Connecticut, June, 1983," *Highway Accident Report*, July, Federal Highway Administration, Washington, DC.

FHWA. 2001. "Chapter 6–Highway bridge replacement and rehabilitation program (HBRRP)," *Local Assistance Program Guidelines*, December 20, Federal Highway Administration, Washington, DC.

FHWA. 2006. "Seismic retrofitting manual for highway structures: part 1–bridges," *FHWA-HRT-06-032*, Federal Highway Administration, Mclean, VA.

FHWA. 2008. *Technical Advisory 5104.29 Load-Carrying Capacity Considerations of Gusset Plates in Non-load-Redundant Steel Truss Bridges*, January 15, Federal Highway Administration, Washington, DC.

FHWA. 2009a. "Load rating guidance and examples for bolted and riveted gusset plates in truss bridges," *FHWA-IF-09-014*, February, Federal Highway Administration, Washington, DC.

FHWA. 2009b. "Connection details for prefabricated bridge elements and systems," *FHWA-IF-09-010*, March, Federal Highway Administration, Washington, DC.

FHWA. 2010a. *National Bridge Inspection Standards, Code of Federal Regulations*, Title 23, PART 650. Subpart C, January 25, 2010, Washington, DC.

FHWA. 2010b. *Technical Advisory 5140.31 Inspection of Gusset Plates Using Nondestructive Evaluation Technologies*, January 29, Federal Highway Administration, Washington, DC.

FHWA. 2011a. "Bridge preservation guide–maintaining a state of good repair using cost-effective investment strategies," *FHWA-HIF-11042*, August, Federal Highway Administration, Washington, DC.

FHWA. 2011b. *Guide for Heat-Strengthening of Damaged Steel Bridge Members*, Federal Highway Administration, Washington, DC.

FHWA. 2012a. *FHWA Bridge Programs NBI Data*, Federal Highway Administration, Washington, DC, accessed from http://www.fhwa.dot.gov/bridge/britab.cfm.

FHWA. 2012b. "Steel bridge handbook–redundancy," *FHWA-IF-12-052-Vol.9*, November, Federal Highway Administration, Washington, DC.

Fisher, J. W. 1984. *Fatigue and Fracture in Steel Bridges–Case Studies*, Wiley, New York, NY.

Kaan, B., Barrent, R., Bennett, C., Matamoros, A., and Rolfe, S. 2008. "Fatigue enhancement of welded coverplates using carbon-fiber composites," *Proceedings of ASCE Structures Congress 2008: Crossing Borders*, April 24–26, Vancouver, Canada.

Khan, M. A. 2010. *Bridge and Highway Structure Rehabilitation and Repair*, McGraw-Hill, New York, NY.

Kim, Y. J. 2011. "Recent advances in maintenance and repair of concrete bridges," *SP-277*, American Concrete Institute, Farmington Hills, MI.

Klaiber, F. W., Dunker, K. F., Wipf, T. J., and Sanders, W. W. Jr. 1987. "Methods of strengthening existing highway bridges," *NCHRP Report 293*, National Cooperative Highway Research Program, Transportation Research Board, Washington, DC.

Klaiber, F. W. and Wipf, T. J. 2000, "Strengthening and rehabilitation," Chapter 50, *Bridge Engineering Handbook*, Edited by W. F. Chen and L. Duan, CRC Press, Boca Raton, FL.

Klein, G. 2008. "Collapse of the Koror–Babeldaob Bridge," *Proceedings, 17th Congress of IABSE. Creating and Renewing Urban Structures. Tall Buildings, Bridges and Infrastructure*, Zurich, Switzerland.

NCHRP. 1980. "Design on secondary highways and local roads rehabilitation and replacement," *NCHRP Report 222*, National Cooperative Highway Research Program, Transportation Research Board, Washington, DC.

New York Times. 2007. "Tanker truck fire collapses bay area overpass," retrieved April 29, from http://www.nytimes.com/2007/04/30/us/30collapse.html?_r=0.

Newman, A. 2012. "Strengthening structural steel beams," *ACSE Webinar Series*, December 12, American Society of Civil Engineers, Reston, VA.

NJDOT. 2007. *Standard Bridge Construction Details*, New Jersey Department of Transportation, Trenton, NJ.

NJDOT. 2009. "Section 9–Bridge deck rehabilitation projects," *Design Manual for Bridges and Structures*, 5th Edition, New Jersey Department of Transportation, Trenton, NJ.

NTSB. 2008. "Collapse of I-35W Highway Bridge, Minneapolis, Minnesota, August 1, 2007," *Highway Accident Report*, NTSB/HAR 08/03; PB2008-916203, November 14, National Transportation Safety Board, Washington, DC.

Ocel, J. M. 2013. "Guidelines for the load and resistance factor design and rating of welded, riveted, and bolted gusset-plate connections for steel bridges," *NCHRP (web only document No. 197)*, Transportation Research Board, National Research Council, Washington DC.

Podolny, W. Jr. 1985. "The cause of cracking in posttensioned concrete box girder bridges and retrofit procedures," *PCI Journal*, 30(2), 82–139.

Poston, R. W. and West, J. S. 2005. "Investigation of the Charlotte Motor Speedway Bridge collapse," *Proceedings of the 2005 Structures Congress and the 2005 Forensic Engineering Symposium*, ASEC, New York, NY.

Radomski, W. 2002. *Bridge Rehabilitation*, Imperial College Press, London, UK.

Rahim, A., Jansen, D., and Abo-Shadi, N. 2006. "Concrete bridge deck crack sealing: an overview of research," *Final Report: F05IR345*, California Polytechnic State University, San Luis Obispo, CA.

Reid, I. L. K., Milne, D. M., and Craig, R. E. 2001. *Steel Bridge Strengthening–A study of Assessment and Strengthening Experience and Identification of Solutions*, WS/Atkins Transportation Engineering Report for Highways Agency, Thomas Relford, London, UK.

Rolfe, S., Matamoros, A., Bennett, C., and Barrett-Gonzalez, R. 2013. "Enhancement of welded steel bridge girders susceptible to distortion-induced fatigue," *TPF-5(189) Project*, March 16, University of Kansas, Lawrence, KS.

Shanafelt, G. O. and Horn, W. B. 1984. *Guidelines for Evaluation and Repair of Damaged Steel Bridge Members*, NCHRP Report 271, National Cooperative Highway Research Program, Transportation Research Board, Washington, DC.

Shanafelt, G. O. and Horn, W. B. 1985. *Guidelines for Evaluation and Repair for of Prestressed Concrete Bridge Members*, NCHRP Report 280, National Cooperative Highway Research Program, Transportation Research Board, Washington, DC.

Silano, L. G. 1993. *Bridge Inspection and Rehabilitation–A Practical Guide*, Wiley, New York, NY.

Waheed, A., Kowal, E., and Loo, T. 2004. *Repair of Bridge Structural Steel Elements Manual*, Version 1.0, June, Alberta Transportation, Calgary, Alberta, Canada.

Xanthakos, P. P. 1995. *Bridge Strengthening and Rehabion*, Prentice Hall, Upper Saddle River, NJ.

17

Rehabilitation of Strengthening of Orthotropic Steel Bridge Decks

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17.1 Structural Details and Modes of Fatigue Damage

Orthotropic Steel Bridge Decks (OSD) are lightweight compared with reinforced concrete decks and are generally used in long-span bridges. Figure 17.1 shows a typical box girder with OSD. OSD consist of deck plates, longitudinal ribs, transverse ribs, and stiffeners. Figure 17.2 shows the types of longitudinal ribs. Open ribs such as flat plates and bulb plates were widely adopted in the early stage and closed ribs have become common. Joints between deck plates and longitudinal ribs, transverse ribs, vertical stiffeners, connection between longitudinal ribs and transverse ribs are the sites of fatigue damage.

Various type of fatigue cracks have been observed in the OSD (Suganuma et al. 2003; Miki 2003; Miki and Konishi 2009). Figure 17.3 shows the modes of fatigue damage in OSD with longitudinal closed ribs.

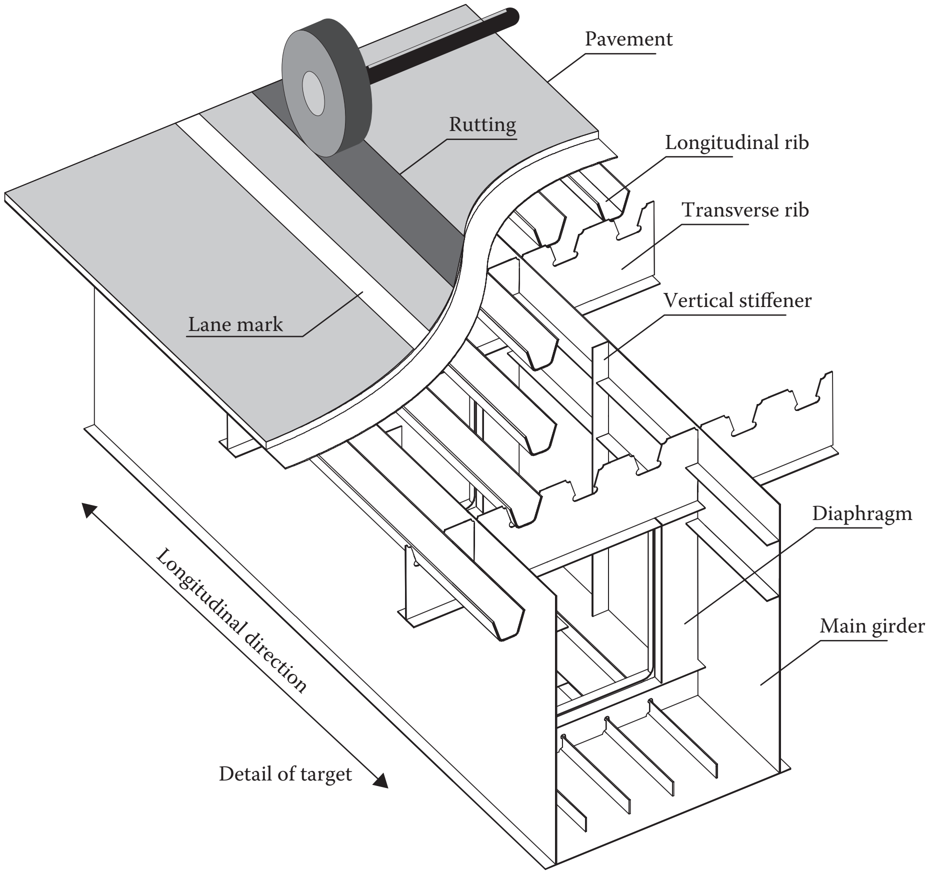


FIGURE 17.1 General structure of box section girder with orthotropic steel bridge decks.

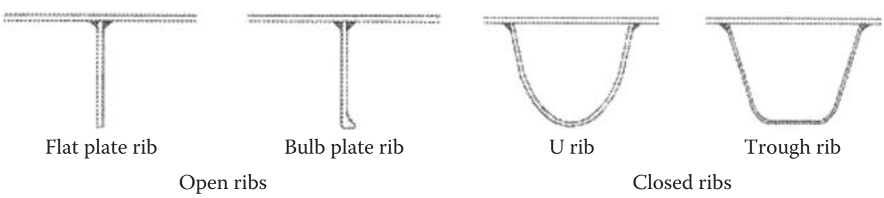


FIGURE 17.2 Types of longitudinal ribs.

Fatigue cracks were observed on welds between butt joint of longitudinal closed ribs (RR: RR1, RR2), longitudinal ribs and deck plates (FR: FR1, FR2), longitudinal ribs and transverse ribs or diaphragms (DS: DS1, DS2; DR: DR1, DR2), deck plates and transverse ribs or diaphragms (FD), vertical stiffeners and deck plates (BA), and web plates and transverse ribs or diaphragms (WD).

Table 17.1 shows the number of OSD spans, which were inspected in the Metropolitan Expressway (MEX) in Japan between 2002 and 2004 with longitudinal rib types, which are about 55% of the OSD spans in MEX. Damage in this table includes all type of fatigue cracks. Regarding OSD spans older than 10 years, 86% of all spans have been inspected once, and 43% of them have fatigue damage. Damage ratio of closed ribs is more than twice of that of open ribs. Considering that bridges with closed-rib OSD have been constructed after the 80s of the twentieth century and are relatively young, closed-rib-type OSD is prone to fatigue damage than open-rib-type OSD. The reason is supposed to be the one side welding of closed rib and out-of-plane bending due to the inclined web of the closed rib.

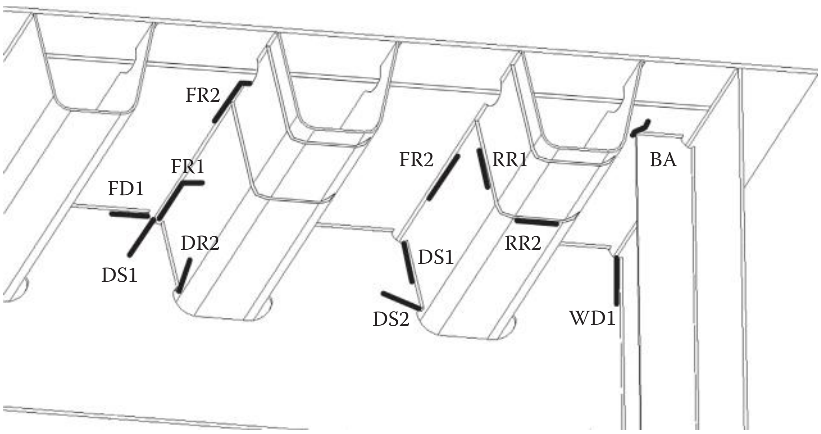


FIGURE 17.3 Modes of fatigue damage in orthotropic steel bridge decks.

TABLE 17.1 Number of OSD Span Inspected between 2002 and 2004

	Years in Service	Number of span (A)	Number of Inspected Span (B)	Damage (C)	Inspection Rate (B)/(A)[a]	Damage Rate (C)/(B)[a]
Plate- or Bulb-Shaped Rib	≤9	101	2	0	0	0
	>10	318	260	82	82	32
	Total	419	262	82	63	31
Trough-Shaped Rib	≤9	409	66	31	16	47
	>10	356	321	215	90	67
	Total	765	387	246	51	64

[a] Values are in percentages.

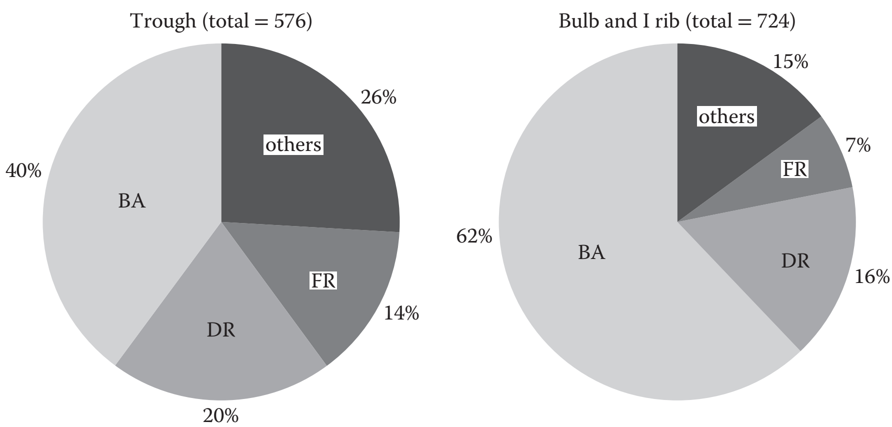


FIGURE 17.4 Statistics of fatigue crack type in the girders of Tokyo Metropolitan Expressway in Japan.

The mode of fatigue cracks depends on the types of ribs. Figure 17.4 shows the statistics of fatigue crack type and number of fatigue cracks observed in girders in MEX. Regarding spans with trough rib, BA-type crack is the greatest proportion and the DR- and FR-type cracks are the second and third proportion. However, DR-type crack is dominant in the spans with flat-plate-type and bulb-plate-type longitudinal ribs.

17.2 Descriptions of Fatigue Cracks and Countermeasures

17.2.1 Butt Welds of Longitudinal Closed Ribs (RR-Type Fatigue Crack)

The cause of this type of fatigue crack is weld defects in butt welds. The weld details of this joint are shown in Figure 17.5. The proper space between adjoining longitudinal ribs and the closeness between ribs and backing plate are essential to secure the proper quality of welds (Gurney 1992).

There are two types of butt-welded joints, field-welded joints and shop-welded joints. In the case of shop-welded joints, welding works are sometimes performed in downward position (Figure 17.6); therefore poor penetration, which results in weld defects such as incomplete penetration and slag inclusion, tends to occur near the deck plate. Fatigue cracks initiate from these defects in the root of welds and propagate to whole section of butt welds. Figure 17.7 shows fatigue cracks (RR1) that initiated from the intersection of the butt welds of closed ribs and the longitudinal welds between deck plate and closed ribs. These cracks often propagate into the longitudinal welds between longitudinal rib and deck plate and penetrate the deck plate.

In the case of field-welded joints, the blocks of longitudinal rib are inserted between neighboring ribs by welding. In such cases, inserted blocks are supported by workers and welded in upward position (Figure 17.8). Gaps between ribs and backing plates, particularly in the corner region of closed ribs, become wide and cause weld defects. Fatigue cracks (RR2) usually initiate from these defects in the corner regions and often propagate to the deck plate and finally penetrate it (Figure 17.9). The gaps between inserted blocks and neighboring closed ribs often deviate from proper distance, specifically, one side becomes narrow and another side becomes wide. Narrow gaps cause incomplete penetration of welds.

FIGURE 17.5 Weld details of butt welds.

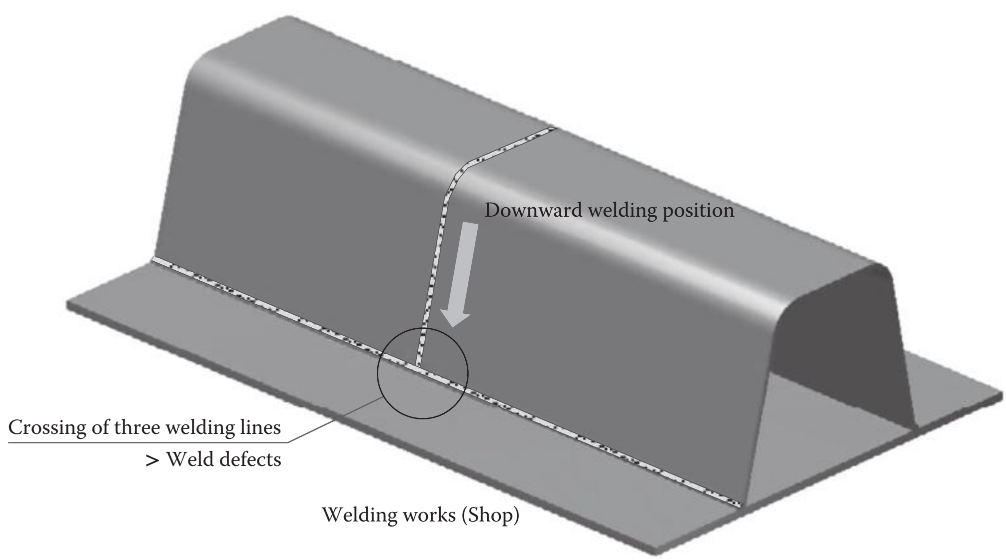


FIGURE 17.6 Welding works, downward welding position.

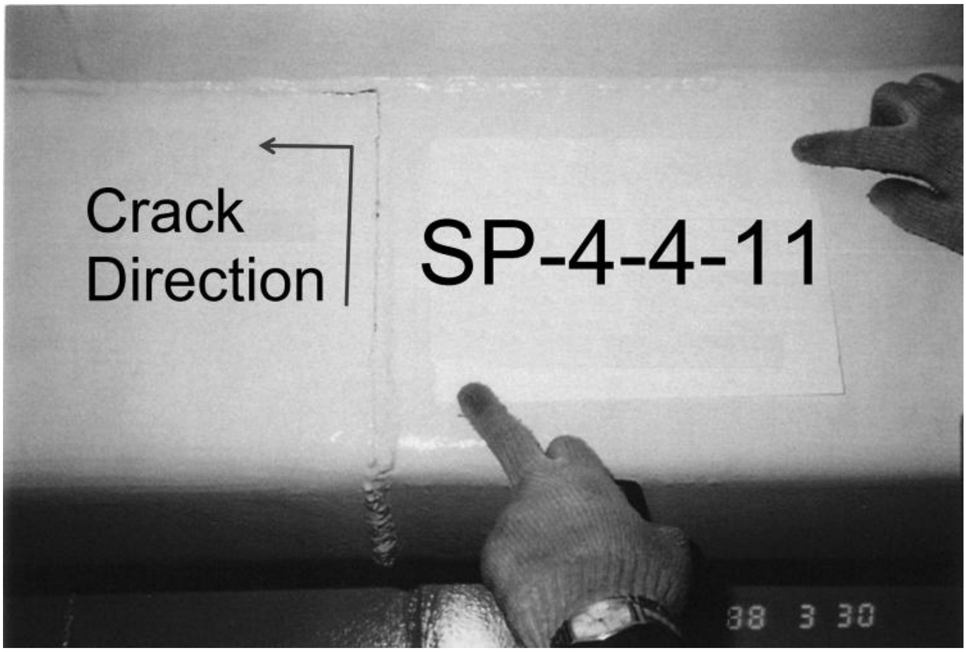


FIGURE 17.7 RR1-type fatigue crack.

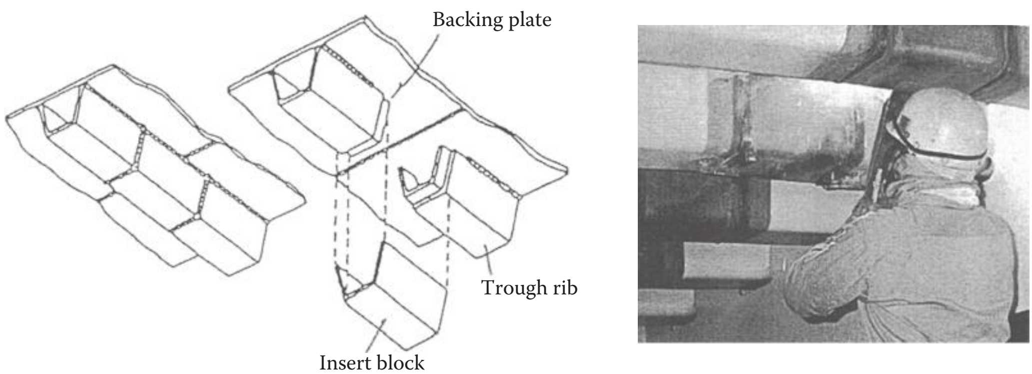


FIGURE 17.8 Welding works, upward welding position.

As a countermeasure, stop holes are drilled at the top of butt welds to isolate fatigue cracks from deck plates. Longitudinal closed ribs are spliced by applying high-strength bolts (Figure 17.10).

17.2.2 Longitudinal Welded Joints between Deck Plate and Rib (FR-Type Fatigue Crack)

There are two types of FR fatigue cracks, FR1 and FR2. FR1 fatigue cracks initiate in the general longitudinal welds between deck plate and web of the closed rib and FR2 cracks at the end of scallop, which is the weld access hole for the transverse butt welds of deck plate in the field connection of OSD panels (Figure 17.3). The cause of FR1 cracks is out-of-plane bending in the deck plates and the web of closed ribs, which is perpendicular to the longitudinal welds between deck plates and closed ribs. Very high stress concentration occurs at the weld root of longitudinal welds due to bending and results in fatigue crack. The cause of FR2 crack is stress concentration due to the notch effect of scallop.

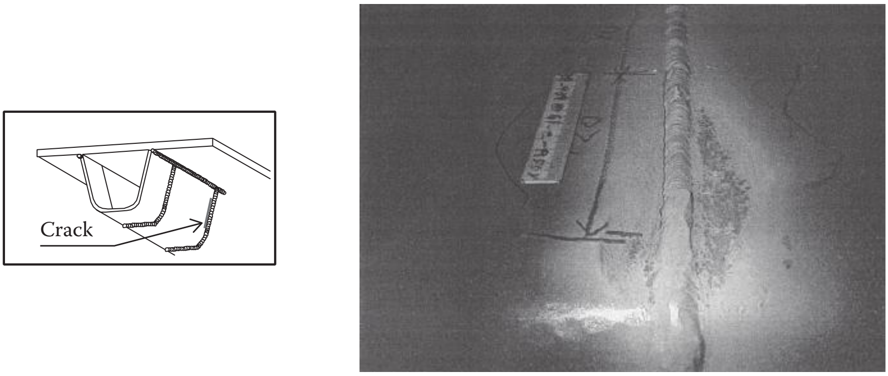


FIGURE 17.9 RR2-type fatigue crack.

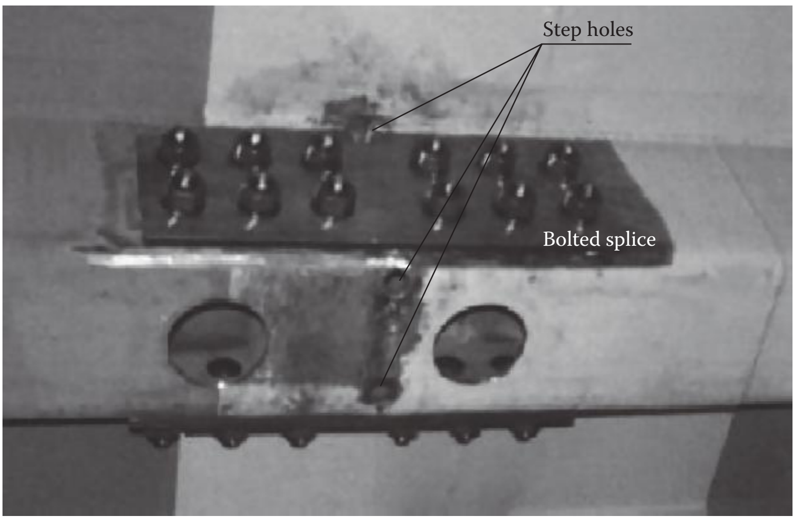


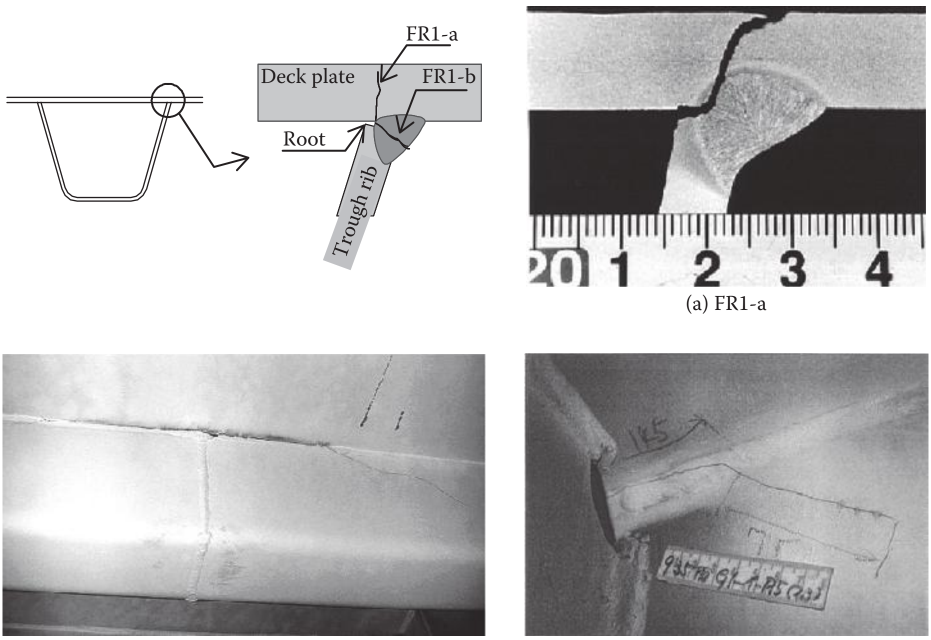
FIGURE 17.10 Countermeasure for RR-type fatigue crack.

There are two types of FR1 fatigue cracks, FR1-a and FR1-b (Figure 17.11). FR1-a-type cracks initiate from the weld root of longitudinal welds and penetrate the deck plate. Finally, this fatigue crack cuts the deck plate and causes the drop-down of the deck. It is very difficult to detect this type of crack because of the wearing surface on the deck plates.

FR1-b-type cracks also initiate from the root of the longitudinal rib, but penetrate the welded bead. This type of crack propagates parallel to the weld bead first and changes direction downward into the web of closed rib afterward. This type of crack sometimes makes new branches at the corner and branching cracks propagate into the deck plate also. Figure 17.11c shows FR1-b-type crack that initiated at the crossing point of longitudinal welds and butt welds. The butt welds in this case were shop-welded, but the penetration of longitudinal welds in the vicinity of crossing point was not adequate.

17.2.2.1 FR1-a Crack (Deck Plate through Crack)

Figure 17.12 shows the full-sized OSD specimen to study the behavior of FR1-a-type fatigue cracks by applying a wheel-moving fatigue test system (Miki et al. 2005; Ono et al. 2005). Figure 17.13 shows a general view of wheel-moving fatigue test. Two wheels of 69 kN (15.51 kip) move cyclically within the region of 3 m.



(a) FR1-a

(b) FR1-b

(c) FR1-b

FIGURE 17.11 Two modes of FR1-type fatigue cracks (FR1-a, FR1-b).

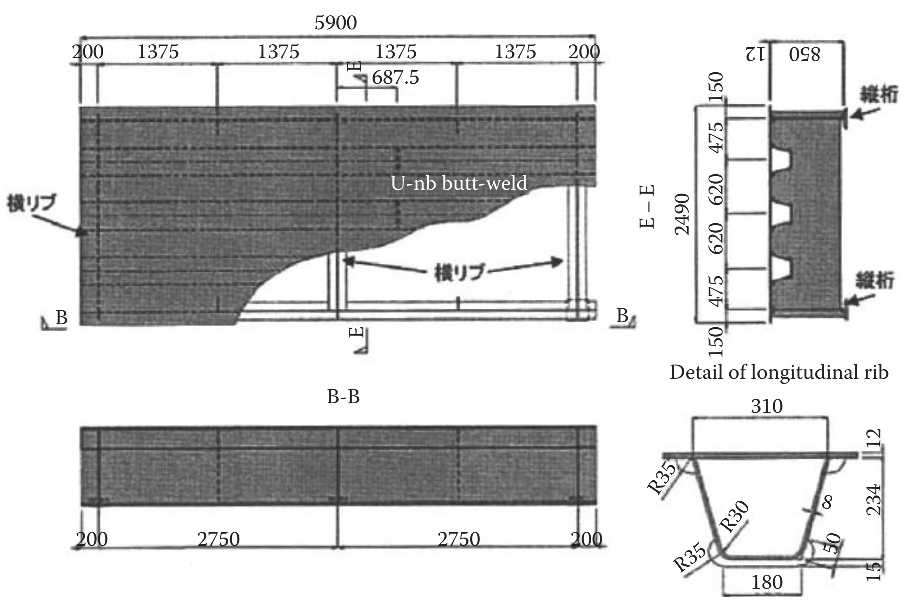


FIGURE 17.12 Full-size specimen to study FR1-a-type fatigue crack.

This specimen includes three longitudinal closed ribs and three transverse ribs. The stiffness of stringers of both sides is adjusted to simulate stresses in the connections between a longitudinal rib and a transverse rib at the center of specimen. The loading position is determined from the results of static loading tests and finite element method (FEM) to produce the maximum stress at the root of longitudinal welds. The initiation and propagation of fatigue cracks were observed by applying ultrasonic test and stress measuring.

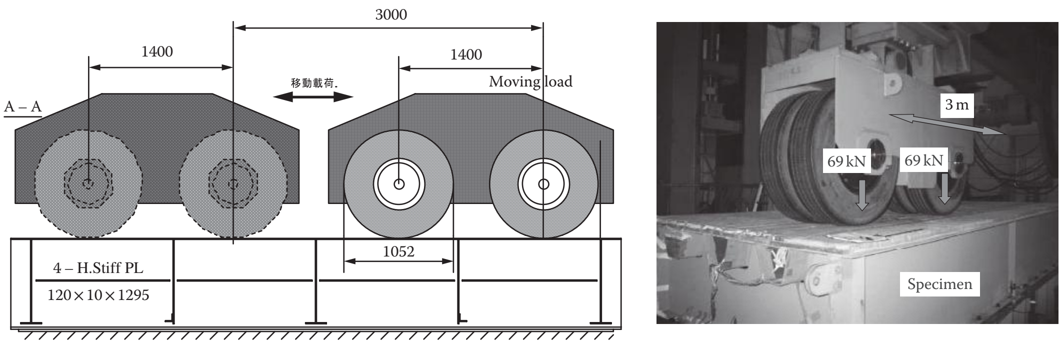


FIGURE 17.13 General view of wheel-moving fatigue test.

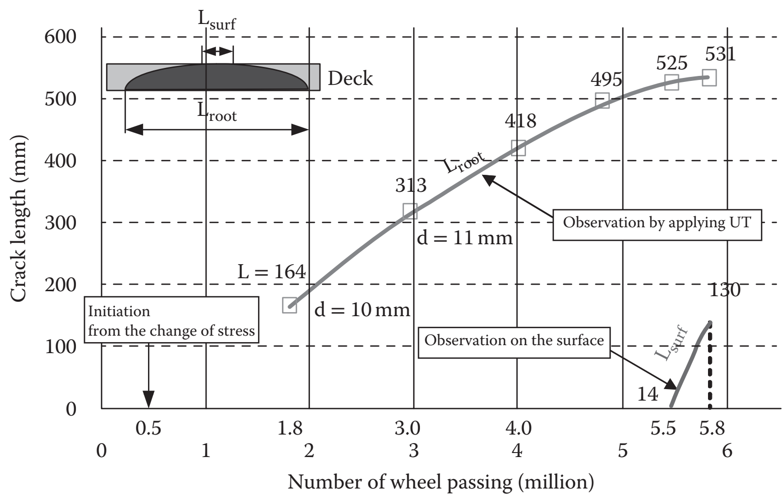


FIGURE 17.14 Fatigue crack initiation and propagation.

Figure 17.14 shows the observation results of fatigue crack initiation and propagation with loading cycles. The stress measured on the back surface of deck plate started to drop after 0.5 million loading cycles, which means fatigue cracks initiated near strain gauges. A 164 mm (6.46 in.) long embedded fatigue crack was detected by ultrasonic tests at 164 million loading cycles. A 525 mm (20.67 in.) long fatigue crack appeared on the deck surface at 5.5 million loading cycles. This fatigue test indicates the very important fact that fatigue cracks initiated in the early stage of fatigue test at the root of longitudinal welds in the connection between longitudinal and transverse ribs develops into long cracks inside the deck plate and finally appears on the surface of the deck plate.

After the fatigue test, all welds root were cut out and exposed. Figure 17.15 shows the exposed surface of an FR1-a-type fatigue crack, which revealed many fatigue cracks initiated along weld root and coalesced to long cracks.

To avoid subsiding of road surface, early stage detection of FR1-a type fatigue cracks is essential. The MEX developed a semiautomatic ultrasonic test system (SAUT) and applied it intensively in OSD bridges where truck traffic is heavy. This system is based on the application of ultrasonic probe of 70 degree angle wave. Fatigue cracks of 6 mm (0.24 in.) deep can be detected by this system. Figure 17.16 shows the general view of inspection using SAUT and an example of inspection results. An FR1-a crack of 360 mm (14.17 in.) length is detected by this system. This result indicates long fatigue cracks exist along the root of longitudinal welds. However, more accurate nondestructive inspection systems are needed to be developed to detect early stage fatigue cracks with smaller size.

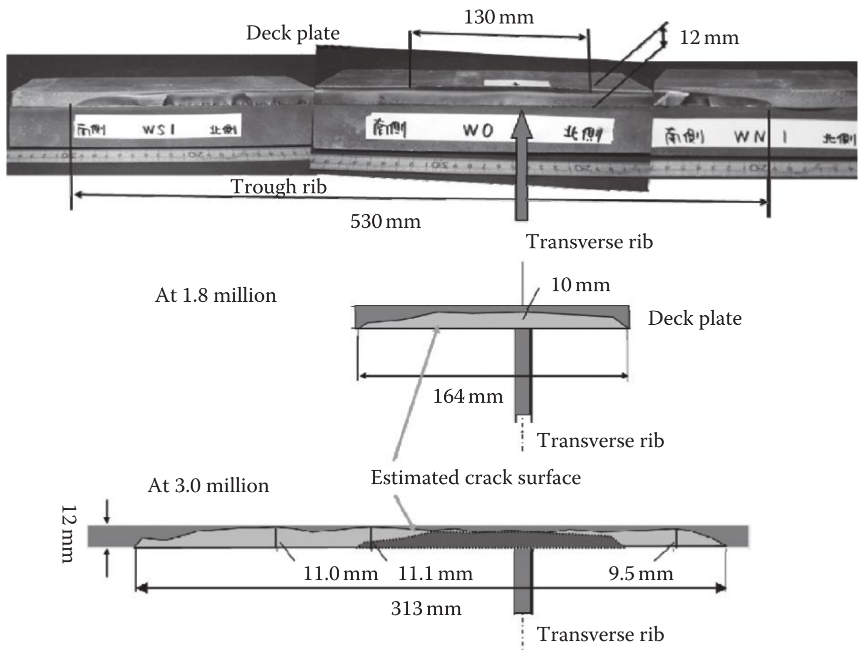


FIGURE 17.15 Exposed surfaces of FR1-a-type fatigue crack.

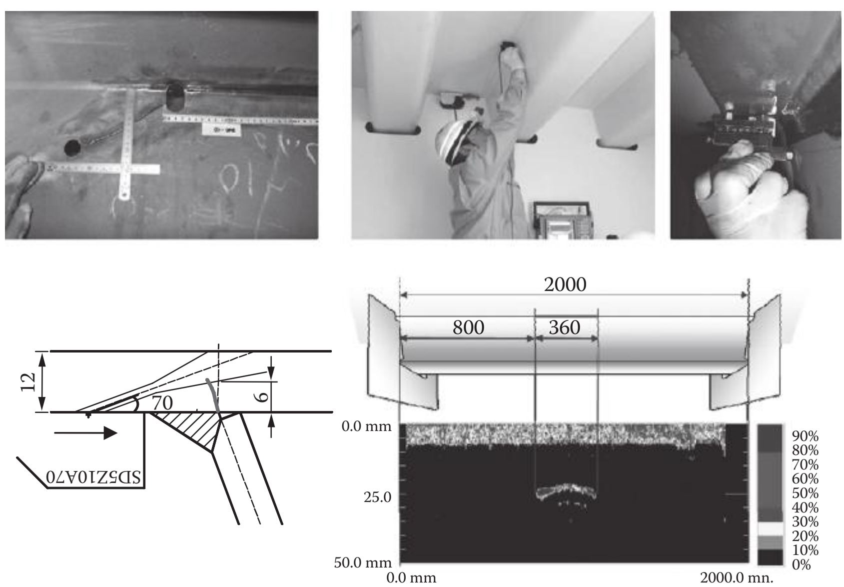


FIGURE 17.16 Inspection by using semiautomatic ultrasonic test system.

If this type of cracks are found to have penetrated the deck plate, a patch plate is soon attached on the deck plate to prevent extension (Figure 17.17). But this countermeasure is not always effective and only temporary. It is because this patch plate is bolted to the deck plate only outside of closed ribs, the regions between webs are unbolted. In addition, the deck plate is not flat and the patch plate touches the deck on limited contact area, such as the deck above the weld line of closed rib. Hitting and rubbing sometimes introduces new fatigue problems such as failures of high-strength bolts.

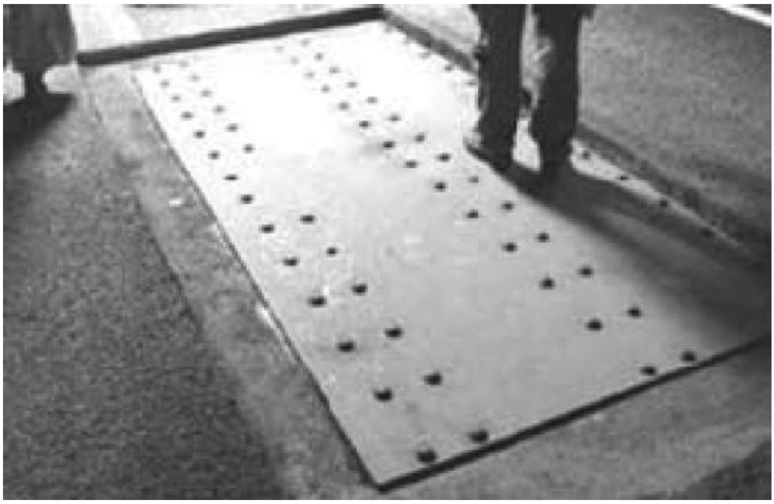
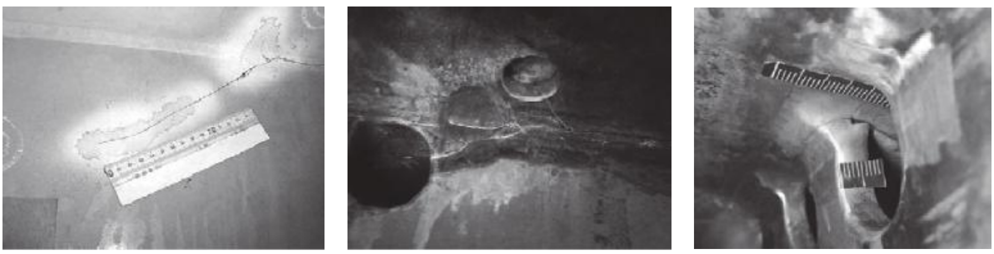


FIGURE 17.17 Plate patching for FR1-type fatigue crack.



(a) (b) (c)

FIGURE 17.18 FR1-b-type cracks in weld bead: (a) Crack changes direction hole for FR1-b; (b) Branching crack; (c) Observation.

17.2.2.2 FR1-b Crack (Weld Bead Cut Crack)

Figure 17.18 shows FR1-b-type fatigue cracks that initiate at the root of longitudinal welds and propagate into willed welds along weld root and then come out on the surface of welds. FR1-b-type cracks have a relatively early stage of life when compared with FR1-a-type cracks. After fatigue cracks appear on the surface, they propagate along the weld bead first and move downward into the web of the closed rib (Figure 17.18a).

Figure 17.18b shows stop holes applied to the tips of FR1-b fatigue cracks. In case a fatigue crack changes its direction and propagates into the web of the closed rib, a hole is applied at the turning corners. It is necessary to check if deck through cracks are branching and propagating to the deck from the corner exists (Figure 17.18c). This deck through crack propagates to transverse direction in deck plate, so additional stop holes will be required at the tips of deck cracks.

After about 1-year service, new fatigue cracks initiated from the edge of the stop hole (Figure 17.19). The local decreasing of stiffness due to a stop hole introduces very high stress concentration under the transverse loading because of the combination of bending moment and shear force. In addition, exposed weld roots also cause high stress concentration. Then, the fatigue strength of this stop hole becomes very low.

When this type of cracks grow longer, the existing closed ribs are removed and new ribs are attached to the deck. New ribs were connected to the deck plate and existing closed ribs by applying high-strength bolts (Figure 17.20). Because the connections between existing and new ribs may become new stress-raising spots, weak spots for fatigue, connection details were designed to lower the stress concentration at the edge of existing and new ribs.

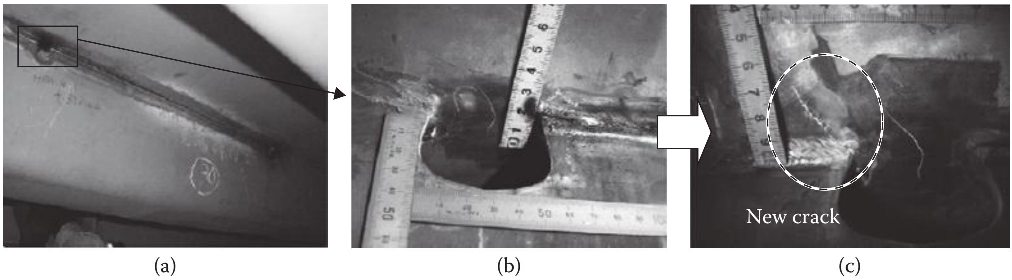


FIGURE 17.19 Observation of the surface of stop hole surface after 1 year, new fatigue cracks initiated at the edge of stop hole and on the surface of deck plate: (a) Stop hole for FR1-b; (b) Scallop to check cracks in deck; (c) Reappearance of crack.

FIGURE 17.20 Partial replacement of longitudinal rib.

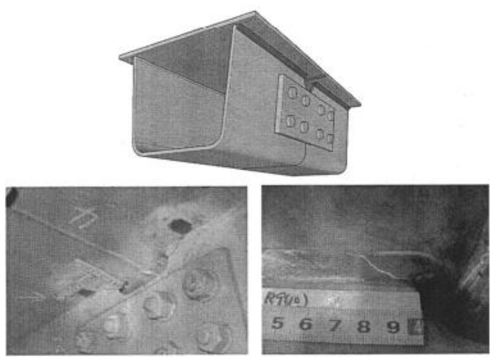


FIGURE 17.21 FR2-type fatigue crack.

17.2.2.3 FR2-Type Crack from the Edge of Scallop

Figure 17.21 shows FR2-type crack initiated from the toe of boxing welds at the edge of scallop. This is a typical transverse field connection detail in which the webs of the closed rib are spliced by using high-strength bolts and deck plates are connected by applying welds with full penetration. In order to perform this full penetration welds by applying soft backing material, and to perform x-ray examination, a space is necessary and a round cutoff called scallop was adopted. However, this scallop causes stress concentration at the edges. Because of sudden change of bending stiffness of deck plate, stresses due to transverse loads are very high and result in fatigue damage.

Wheel load also induce bending in the web of longitudinal closed rib because the deck plate and web of the closed rib is not at right angle. This bending also induces high local stress at the edge of scallop in the web of the closed rib.

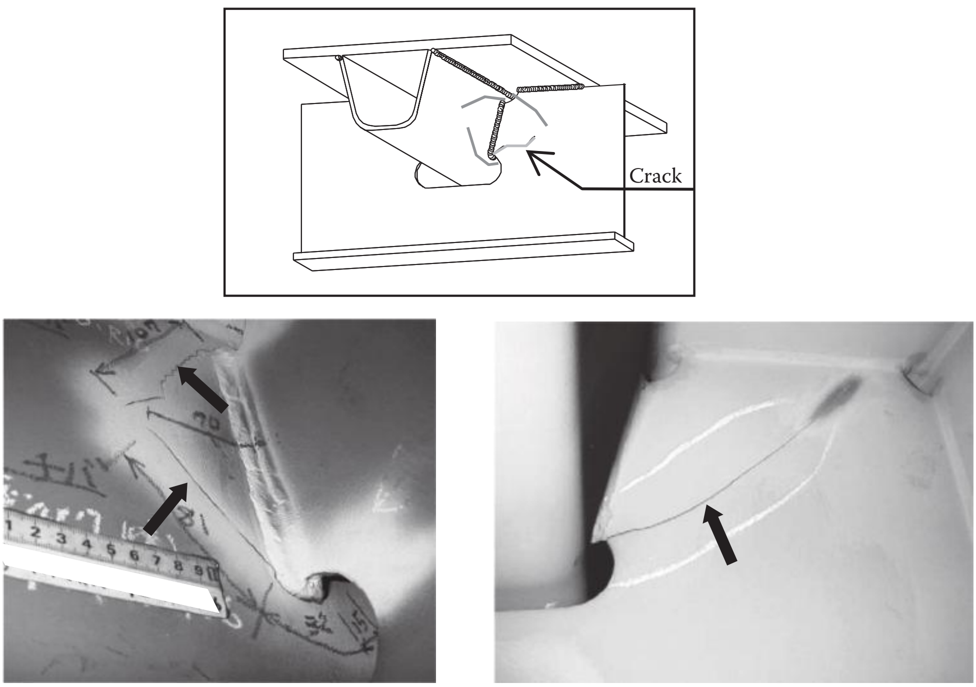
In order to reduce the stress concentration, shorter length of space is better. The length was 150 mm (5.91 in.) originally, but it is shortened to 75 mm (2.95 in.) in recent practice.

17.2.3 Connection between Longitudinal Rib and Transverse Rib (DR- and DS-Type Cracks)

Fatigue cracks initiated in the connection between longitudinal ribs and transverse ribs are shown in Figure 17.22. There are slits at the lower end and upper ends of transverse ribs in the connection. Fatigue cracks initiated both in the upper end and in the top end of connections. There are two modes of cracks, one is the crack that initiates from the weld toe on the web of the transverse rib and propagates into the web of transverse rib (DR) and the other is the crack that initiates from the weld toe on the trough rib and propagates into the closed rib (DS). From the observation of the modes of fatigue cracks, it is clear that stresses in the web of transverse rib are more dominant to fatigue crack initiation and propagation than those in the web of longitudinal ribs.

17.2.4 Welds between Vertical Stiffener and Deck Plate (BA-Type Cracks)

This mode of fatigue crack initiates along the weld toe of boxing weld at the top of vertical stiffener (Figure 17.23). Fatigue cracks initiate both weld toe of fillet welds, vertical stiffener side and deck plate side (BA-1 and BA-2). This BA-type fatigue crack is the most common fatigue crack in OSD spans in MEX. These cracks quickly propagate and penetrate the deck plate. Initiation points are both the toe



(a)

(b)

FIGURE 17.22 Fatigue cracks initiated in the connection between longitudinal ribs and transverse ribs (DR- and DS-type fatigue cracks): (a) DR1, DR2 type fatigue cracks; (b) DS2 type fatigue cracks.

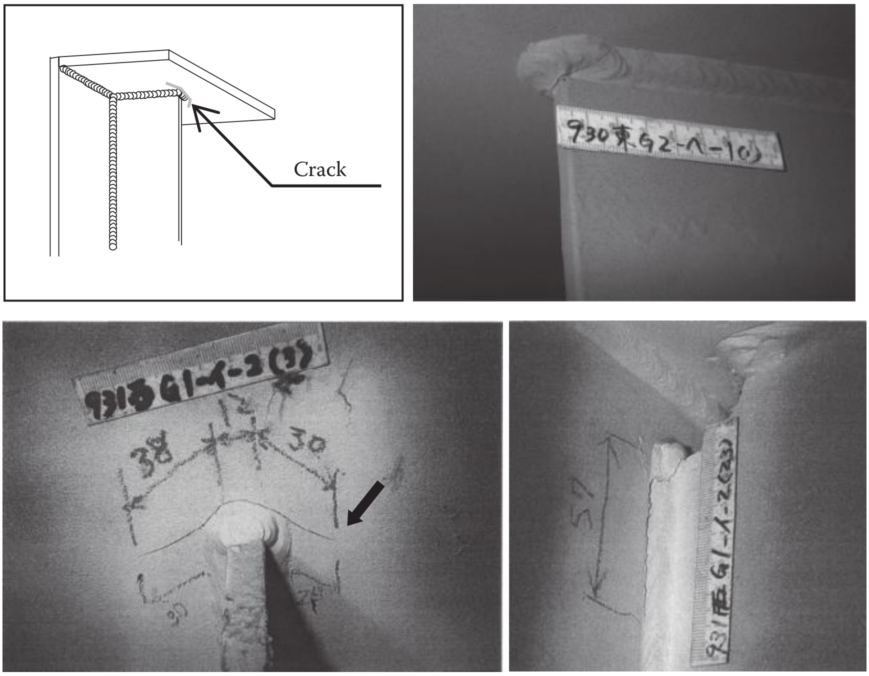


FIGURE 17.23 Fatigue cracks around the top end of vertical stiffener (BA-type fatigue crack).

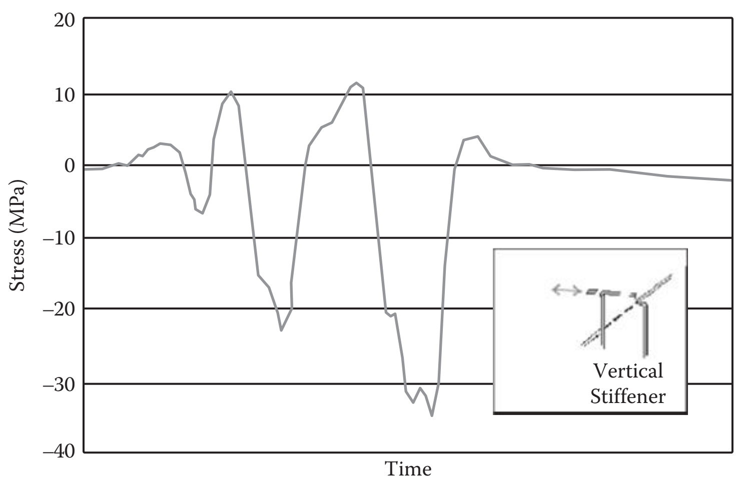


FIGURE 17.24 Change of stress with the passage.

and root of the weld between vertical stiffener and deck plate. Fatigue cracks also are initiated along the boxing welds of fillet welds between vertical stiffener and the web of main girder (BA-3).

Out-of-plane deformation of deck plates due to the direct wheel loads is the major cause of this type of cracks. Figure 17.24 shows the change in stresses on the back surface of deck plate near the top end of vertical stiffener. When the testing truck enters the area surrounded by the web of the girder, adjacent transverse ribs, and a longitudinal rib, stresses because of out-of-bending of the deck plate occur. The maximum stress occurs when the center of the two axes is on the vertical stiffener. The mode of deformations of the deck plate looks like punching shear. It indicates that the increase in stiffness of the deck plate is effective to reduce these stresses that cause BA-type crack.

17.3 Local Stresses in Orthotropic Steel Bridge Deck Structures

17.3.1 Model for Study

Local stresses that cause fatigue damage in OSD are studied in the cases of Maihama Bridge in Tokyo Metropolitan Expressway. Figure 17.25 shows the general view of the bridge. Figure 17.26 is the model for FEM, three-step submodeling analyses were performed and solid elements were applied to evaluate local stresses. Figure 17.27 shows a testing truck used in the field measurements.

The stiffness of asphalt pavement may affect stresses in OSD. The asphalt pavement in Japan usually consists of two layers, which are the goose asphalt mixture of 40 mm (1.57 in.) thickness and the asphalt concrete of 40 mm (1.57 in.) thickness on the surface. Figure 17.28 shows measured stresses on the surface of bottom flange of longitudinal closed rib and temperatures. The stresses strongly depend on the temperature, the value of stress is about 35 MPa (5.08 ksi) at 35°C and 10 MPa (1.45 ksi) at 10°C.

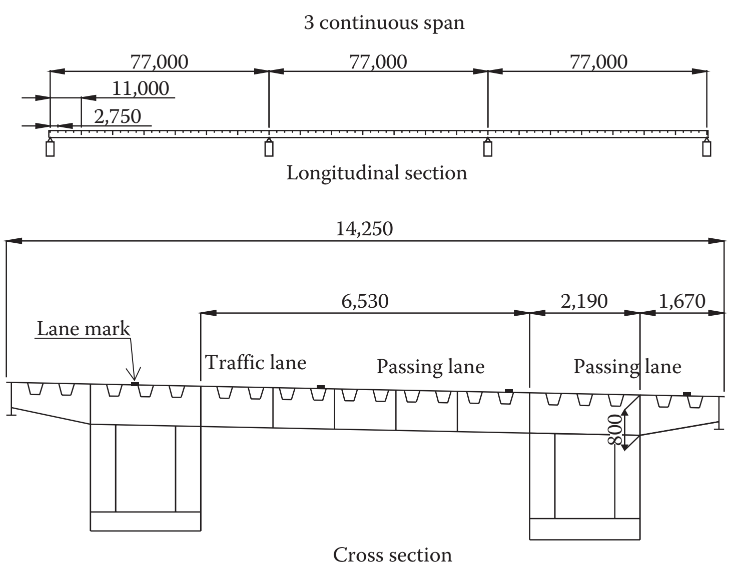


FIGURE 17.25 General view of Maihama Bridge.

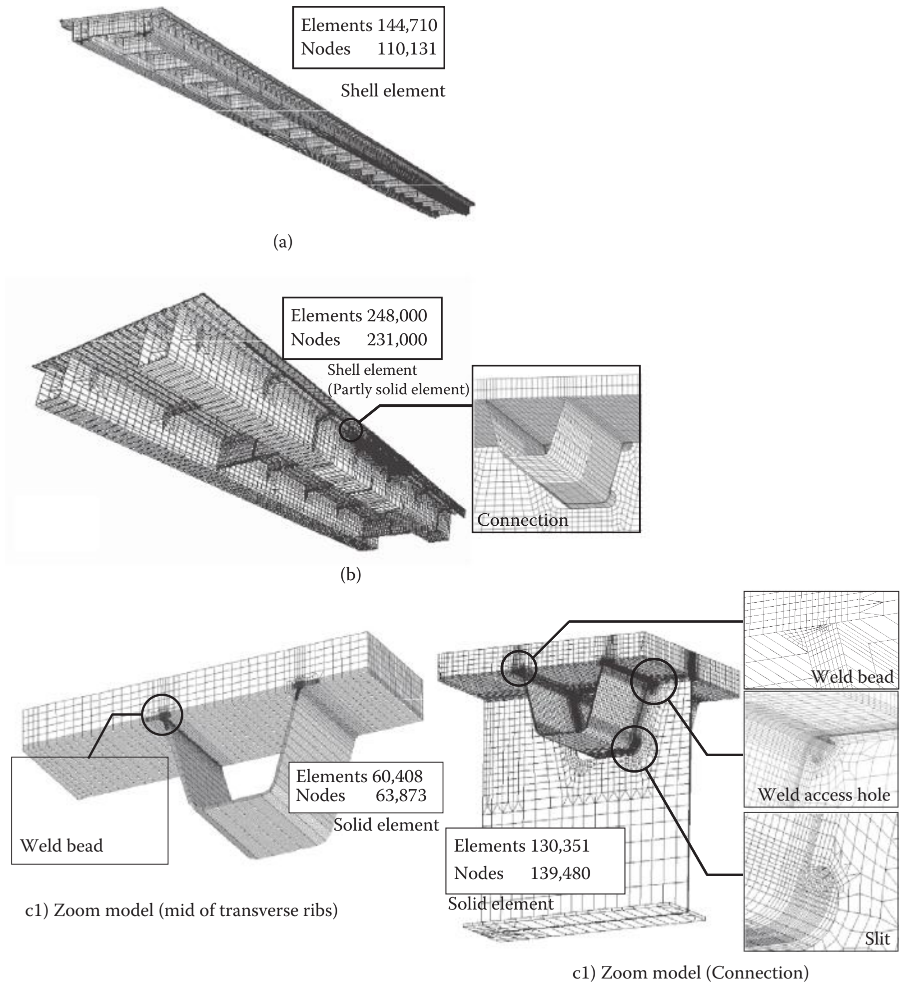


FIGURE 17.26 Finite element method models: (a) Whole model; (b) Detail model.

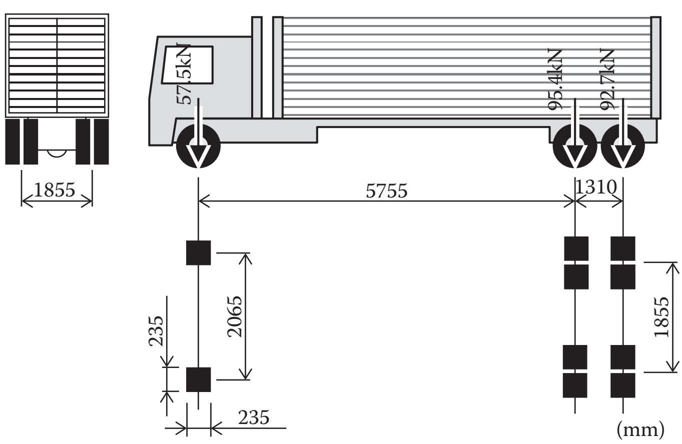


FIGURE 17.27 Test trucks for field measurement.

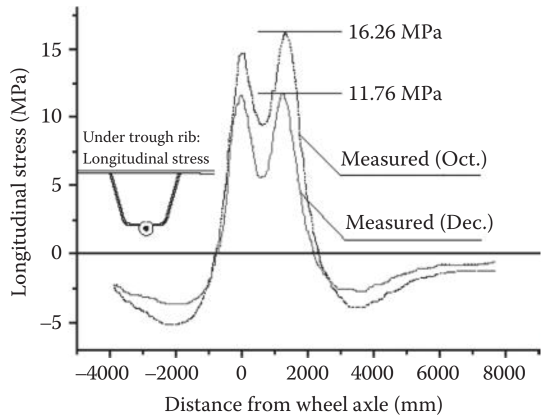


FIGURE 17.28 Measured stresses on the bottom flange of longitudinal rib under the same load in different temperatures.

These differences resulted from the change in the stiffness of the wearing surface. The value of 1500 MPa (217.6 ksi) corresponding to 20°C was assumed as the elastic modulus of asphalt pavement in the FEM.

17.3.2 Welded Joints between Deck Plate and Longitudinal Rib (FR-Type Fatigue Cracks)

Figure 17.29 shows deformations of OSD and local stress perpendicular to welds at the root of longitudinal welds under various positions of truck tire. The arrows indicate the center of each tire of double tires of rear axis. The loading condition that double tires step over the web of target rib results in the highest stress of 77.7 MPa (11.3 ksi) at the weld root. Moreover, the loading condition that double tires step over the web of the next longitudinal rib of target rib also results in a high stress of 67.4 MPa (9.8 ksi).

Figure 17.30 shows stresses on the weld root at the center of free span and at the connection under the various loading positions of the rear axis of truck. Stresses in the connection are about 1.5 times higher than those in free span. The results of the observation in actual bridges indicated that the occurrences of FR-type fatigue crack on the web gap at the connection ribs are notably higher than those in the free span region.

17.3.3 Connection between Longitudinal Rib and Transverse Rib (DS- and DR-Type Cracks)

Stresses that may cause DS- and DR-type fatigue cracks were examined. The observation of the modes of fatigue crack initiation and propagation indicates stresses in the web of transverse rib are more dominant than those in the web of longitudinal ribs. Figure 17.31 shows the change of stresses on the web of transverse rib at the lower edge of bottom slit by the passage of wheel loads. Stresses are separated into in-plane, membrane, stress components, and out-of-plane stress components. Stresses at this point are mainly in-plane and the peak stresses are induced when the center of tandem axes passes on the transverse rib.

Figure 17.32 shows deformations of the connection region of OSD when the rear axis passes over the transverse rib, 550 mm (21.65 in.) before the passage and 550 mm (21.65 in.) after the passage. The deck plate with longitudinal ribs rotates on the transverse rib at the passage of truck axis on OSD.

Stresses in the connection region are very sensitive to the location of tires. Figure 17.33 shows the distribution of minimum principal stresses and deformations when double tires are nearly on the

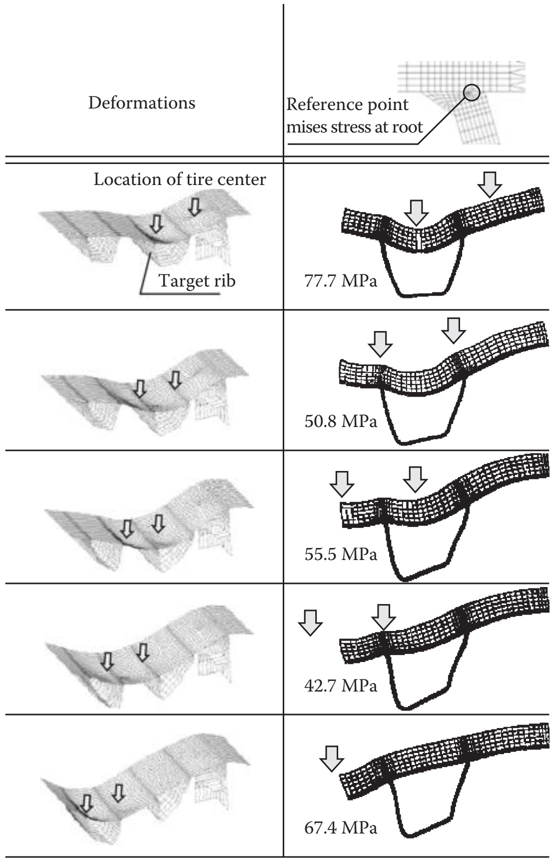


FIGURE 17.29 Deformations and corresponding stresses at the weld root.

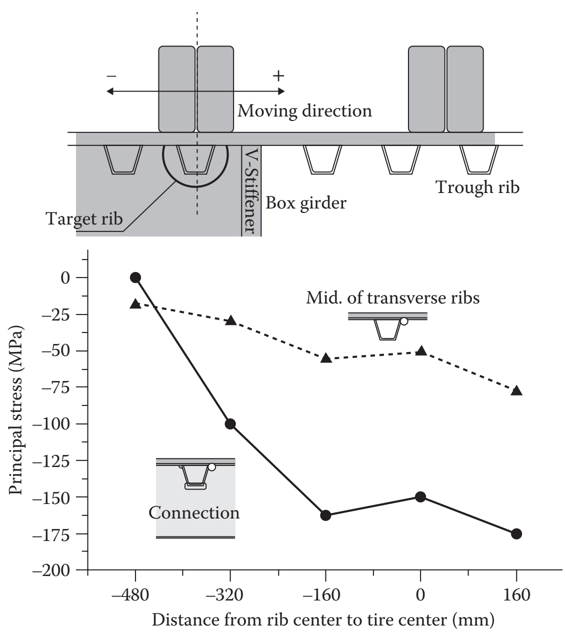


FIGURE 17.30 Stresses on the weld root at the center of free span zone and at the connection.

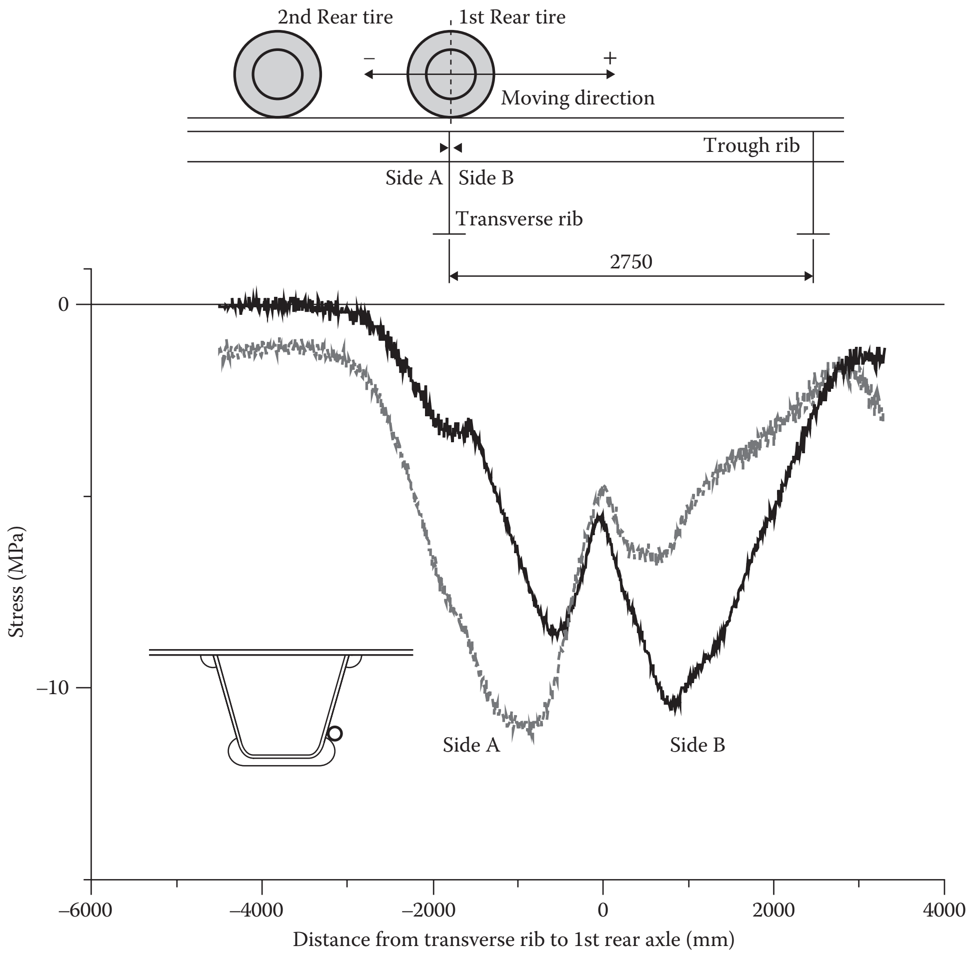


FIGURE 17.31 Change of stresses on the web of transverse rib near the lower edge of connection.

transverse rib, step 1 and 2. Since the downward displacement of longitudinal ribs is constrained by the web of transverse ribs, high stress concentrations induced at the bottom end of welds of connections are the main cause of DS-type fatigue cracks. When tires are on the web of the next rib of the target rib, as in steps 4 and 5, the bottom flange of longitudinal rib moves horizontally. Owing to this shift, tensile stresses are introduced in the lower part of the web of the longitudinal rib on the loading side, and compressive stresses are introduced in the lower part of the web of longitudinal rib. This is called oil canning displacement. The repetitions of oil canning result in DR- and DS-type fatigue cracks.

17.3.4 Deck Plate Near the Top of Vertical Stiffener (BA-Type Cracks)

Figure 17.34 shows the change of stresses on the back surface of the deck plate near the top end of the vertical stiffener. When the load is applied on the area surrounded by the web of girder, adjacent transverse ribs, and a longitudinal rib, stresses attributed to out-of-bending of the deck plate occur. The maximum stress occurs when the center of two axes is applied on the vertical stiffener. The mode of deformations of deck plate that strongly presses the top of the vertical stiffener looks like punching shear. It implies that increasing the stiffness of the deck plate is effective to reduce these stresses that cause BA-type cracks.

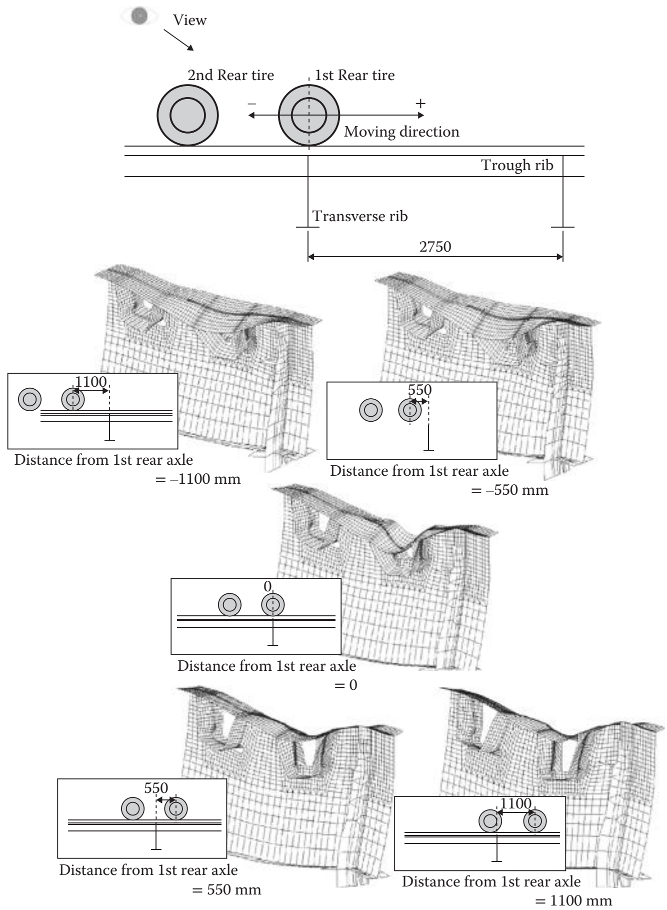


FIGURE 17.32 Deformations of orthotropic steel bridge decks in the connection area.

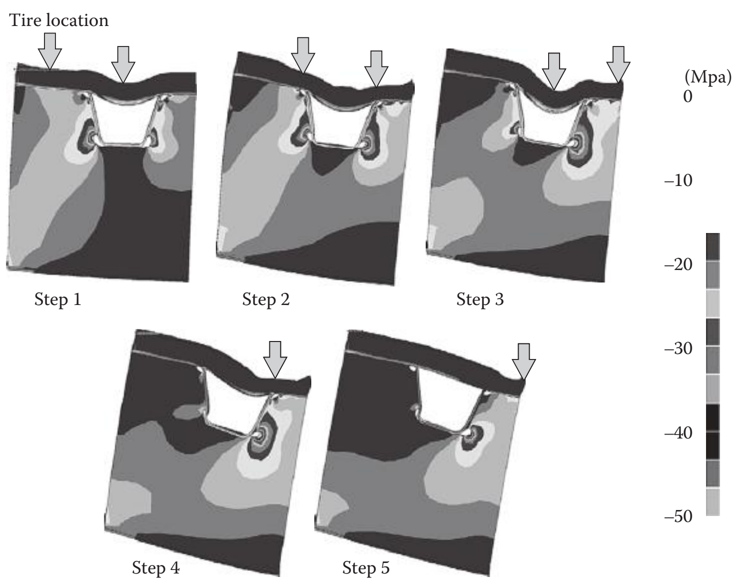


FIGURE 17.33 Deformations and distribution of minimum principal stresses around the slit of connection.

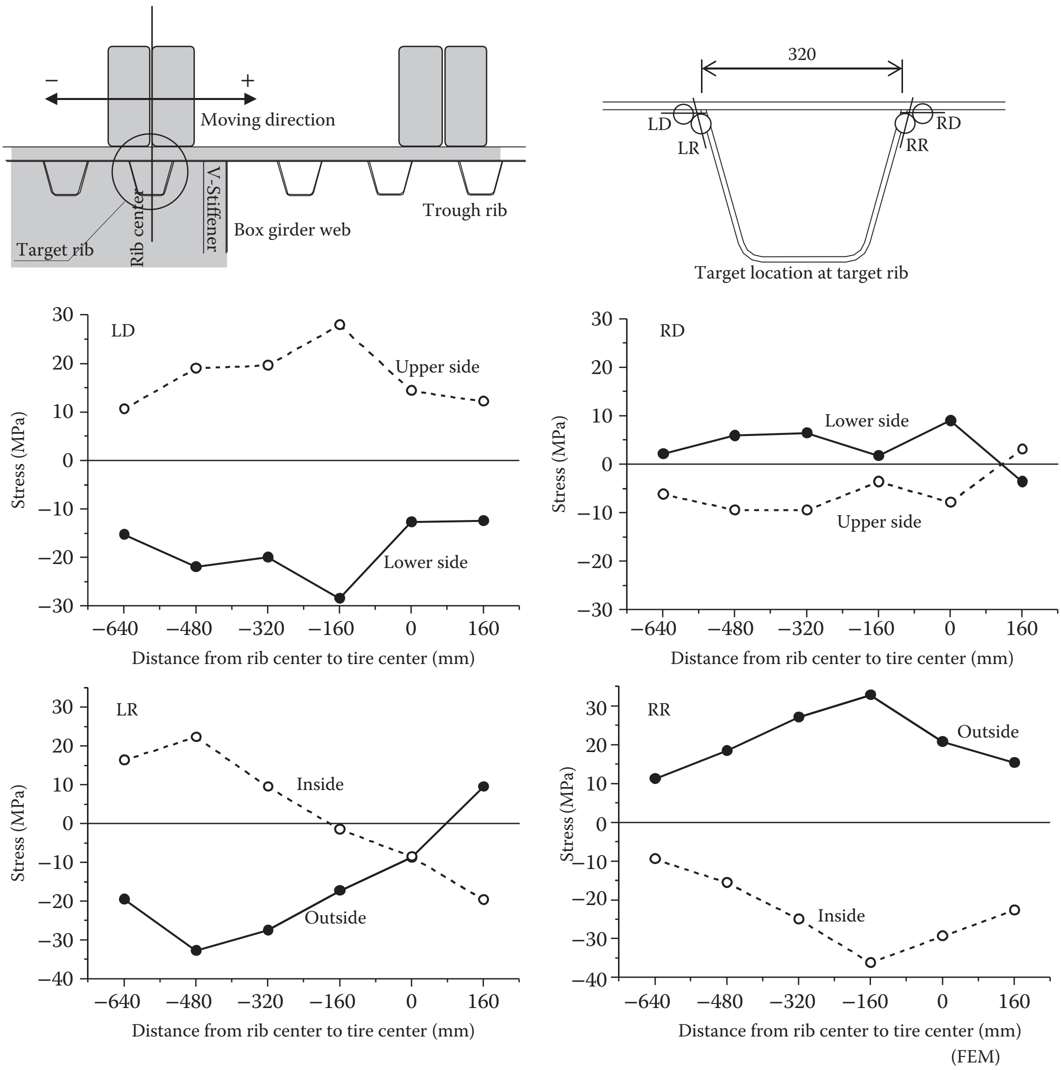


FIGURE 17.34 Variations of stresses due to the shift of loading position (in the transverse directions).

17.4 Increasing the Stiffness of Deck as Retrofitting Works

17.4.1 Finite Element Method

One effective countermeasure against fatigue cracks, which are of FR and BA types, near deck plates is to increase the stiffness of deck plates. Various methods were proposed for retrofitting works (Figure 17.35).

Figure 17.36 shows the configuration and sizes of specimens to examine the performances of the retrofitting methods. The loading stage and testing trucks shown in Figure 17.37 were used.

Figure 17.38 shows the change of stresses on the surfaces of deck plates and longitudinal closed ribs in the free span zone. In the case of no reinforcement, out-of-plane bending stresses occur in the deck plate and the web of the closed rib. However, the application of the steel fiber reinforcement concrete (SFRC) paving method suppresses the out-of-plane bending of deck plate and measured stresses are almost zero. The method of filling concrete inside the closed rib is also effective to reduce stresses in the concrete fill zone; however,

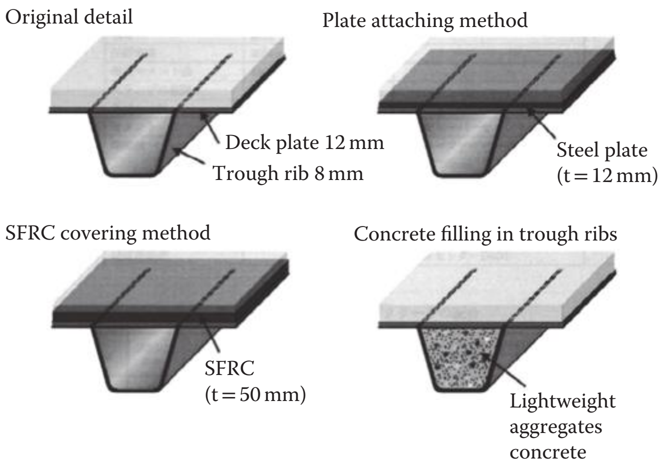


FIGURE 17.35 Proposed methods to increase the stiffness of deck plate as retrofitting measures.

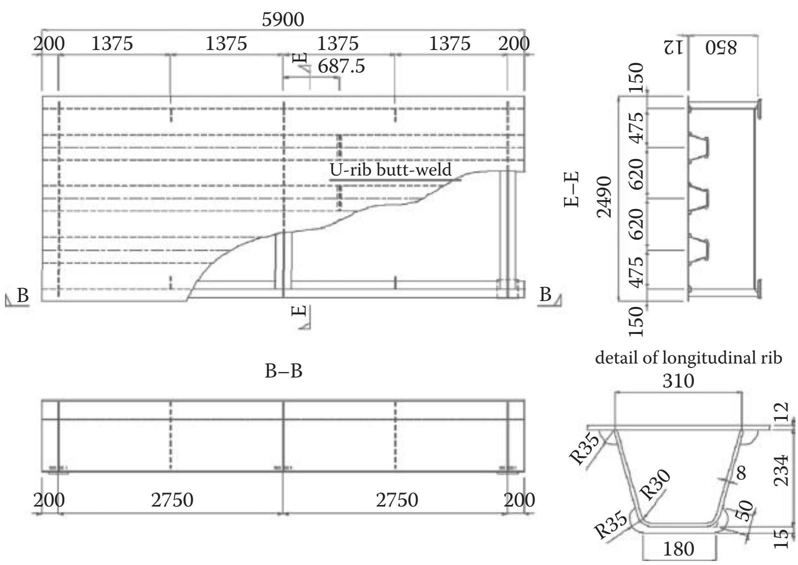


FIGURE 17.36 Full-size specimens to examine the performance of retrofitting methods.

stresses on the outside of the rib increase. The local increase of stiffness of the closed-rib zone results in this behavior. The beneficial effect of steel plate-attaching method is quite low compared with other methods.

Figure 17.39 shows the efficiencies of each method on stress reduction in the connection between longitudinal closed-rib and transverse rib zone. The application of SFRC paving reduces the stresses about 50% in the upper scallop zone and the lower slit zone. Plate-attaching method and concrete filling in the closed-rib method result in an increase in stresses in the lower slit zone.

Fatigue performances of SFRC paving method were examined by applying wheel-moving fatigue testing system. Stresses on the surface of deck plate decrease drastically (Figure 17.40).

From this study, SFRC paving method is the best for retrofitting of existing OSD. In order to apply this method to existing structures, it is necessary to remove the existing asphalt pavements and then place SFRC (Figure 17.41). All fatigue cracks will be removed and repaired by welding at the same time.

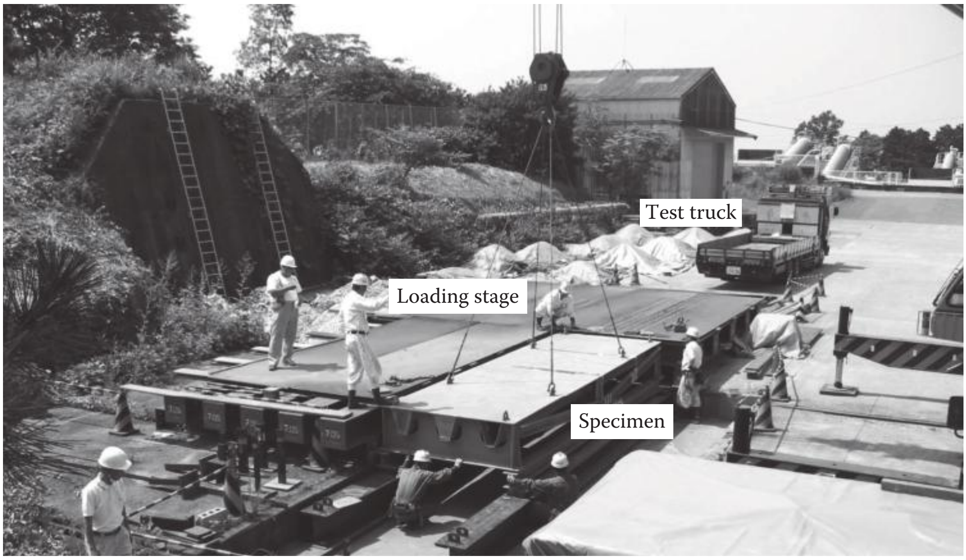


FIGURE 17.37 Loading stage and test truck.

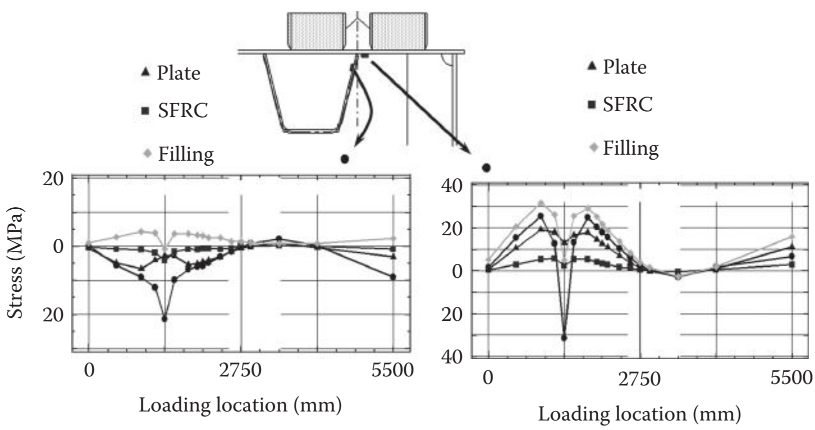


FIGURE 17.38 Stresses before and after retrofitting works.

17.4.2 Application of Steel Fiber Reinforcement Concrete Paving Method as a Preventive Measure

SFRC paving method was applied to a new bridge as a preventive measure (Miki et al. 2007) Yokohama Bay Bridge is a cable-stayed bridge of double decks (Figure 17.42). The total length is 860 m and the center span is 460 m. The upper deck was opened in 1989 as Tokyo Metropolitan Expressway and the lower deck was opened in 2004 as National Highway 357. Because of fatigue damage in the upper OSD, the pavements of lower deck were changed to composite pavements as a preventive measure. The original design was asphalt pavements of 80 mm (3.15 in.) thickness, but these pavements were changed to the SFRC pavements of 50 mm (1.97 in.) (Figure 17.43). The 30-mm-thick (1.18-inch-thick) layer of asphalt pavements was planned to be placed after the examination of performances of this deck. Figure 17.44 shows the results of stress measurements, before and after application of this new pavement system. These results also indicate the beneficial effects of SFRC paving method.

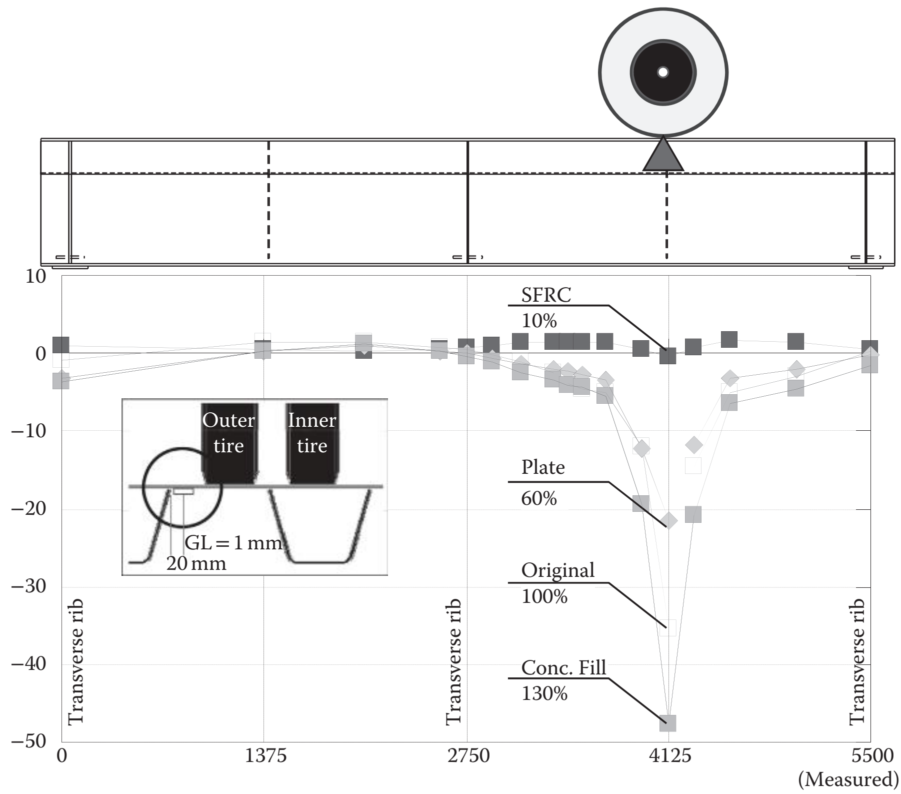


FIGURE 17.39 Effects of each retrofitting work.

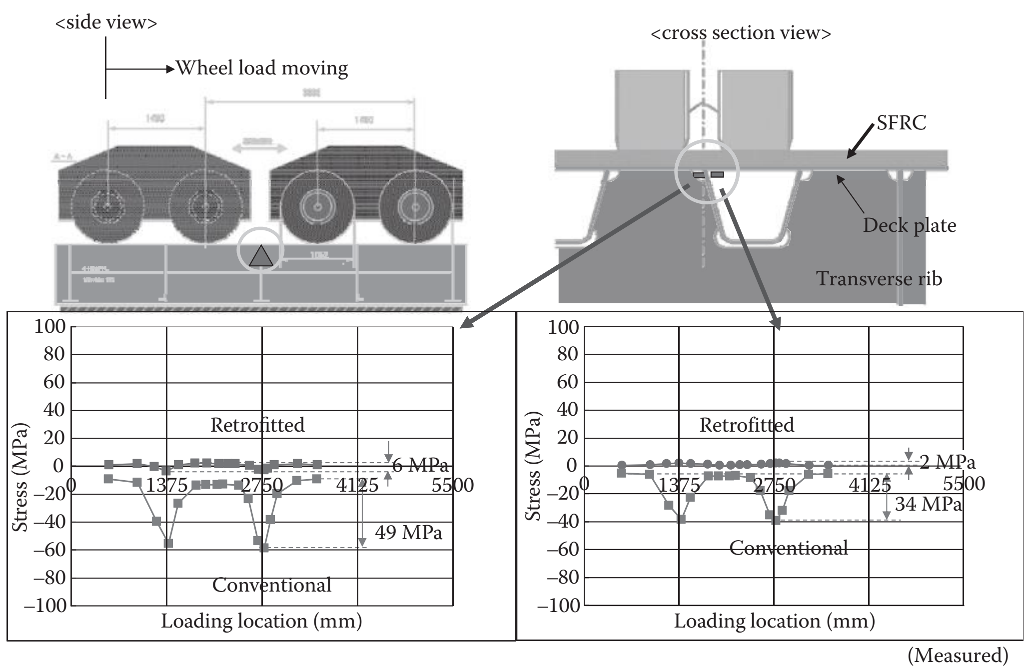
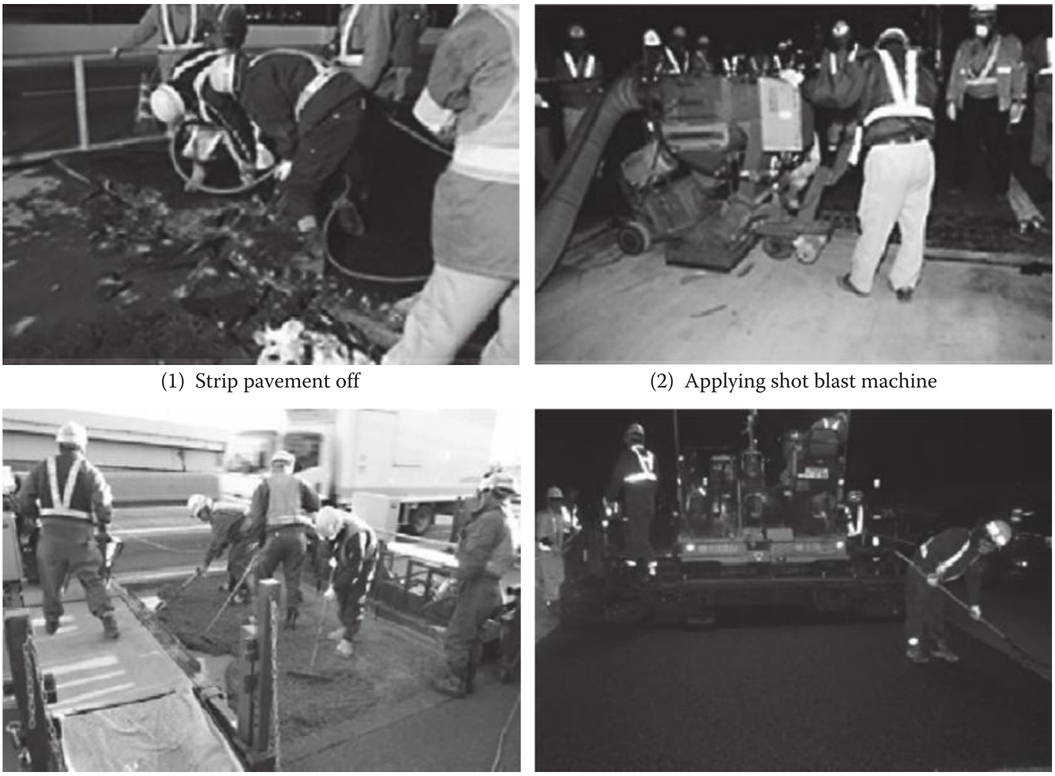


FIGURE 17.40 Stresses on the back surface of deck plate (effect of retrofitting work).



(1) Strip pavement off

(2) Applying shot blast machine

(3) Applying SFRC

(4) Covering with asphalt pavement

FIGURE 17.41 Retrofitting works by applying steel fiber reinforcement concrete.

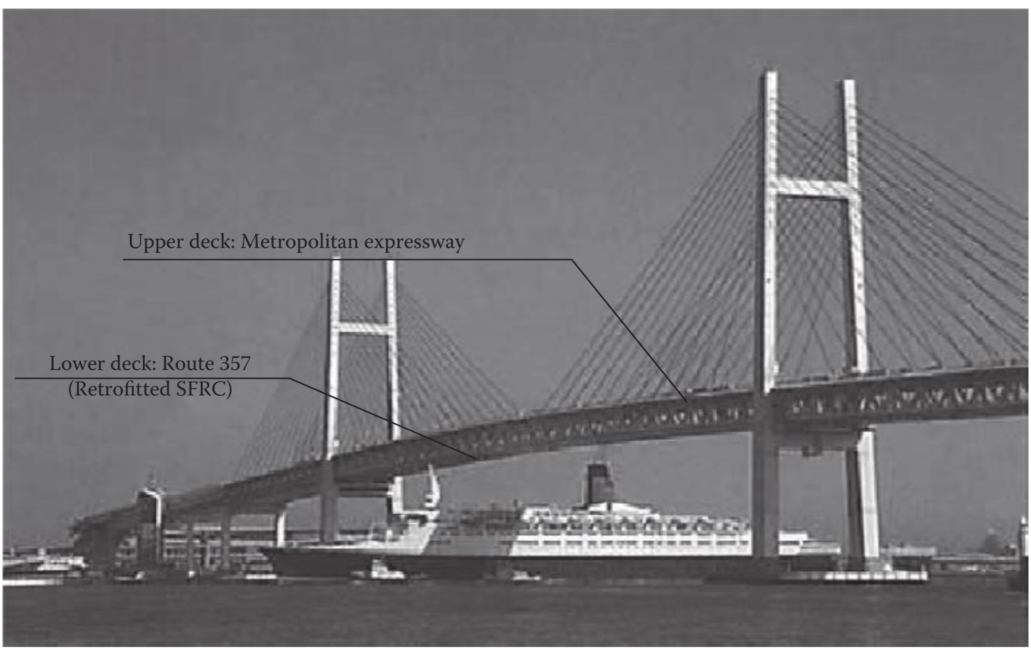


FIGURE 17.42 Yokohama Bay Bridge.

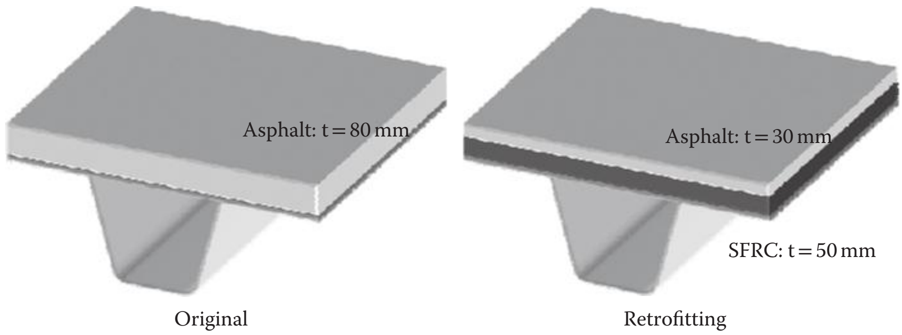


FIGURE 17.43 Modification of asphalt pavements to steel fiber reinforcement concrete pavements in the lower deck of Yokohama Bay Bridge.

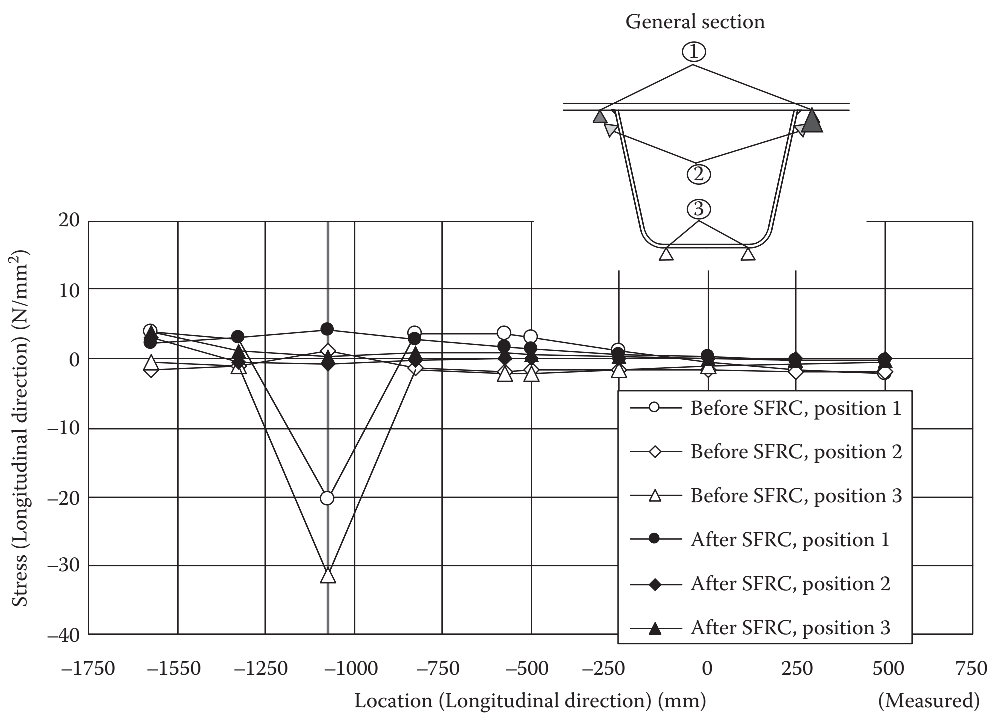


FIGURE 17.44 Measured stresses before and after the placement of steel fiber reinforcement concrete.

17.5 New Orthotropic Steel Bridge Decks for Tokyo Gate Bridge

17.5.1 Study Procedure

Figure 17.45 shows the general view of Tokyo Gate Bridge that was opened on February 12, 2012. The main bridge is three span continuous truss girders and approach span is box section girders. The total length is 2618 m (876 + 160 + 440 + 160 + 982 m) [8589 ft (2874 + 525 + 1444 + 525 + 3222 ft)].

The traditional OSD in Japan consists of 12-mm-thick (0.47-inch-thick) deck plates, 300- to 320-mm-wide (11.81- to 12.60-inch-wide) longitudinal closed ribs, and 2000–2500 mm (78.94–98.43 in.) interval of transverse ribs. OSD of this bridge was newly designed to achieve high fatigue resistance under the condition of the same fabrication cost as traditional OSD. In order to improve fatigue performance in deck plate

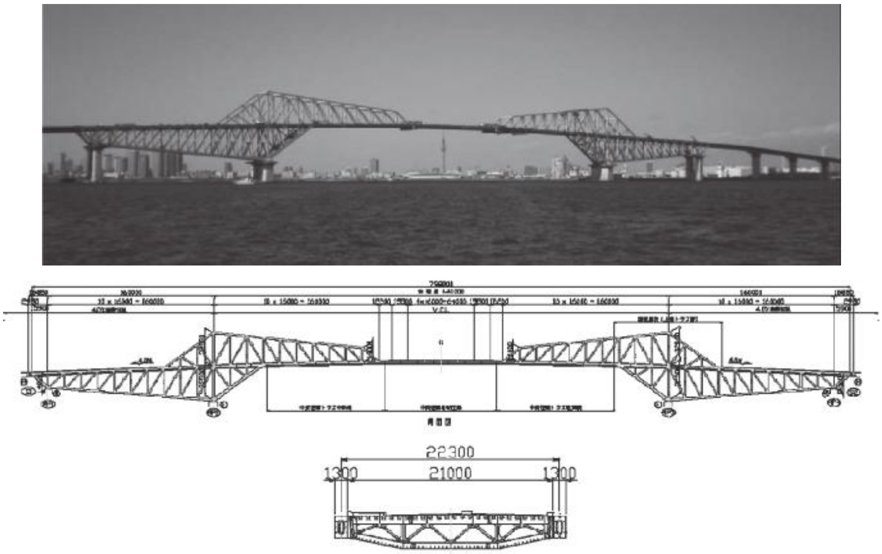


FIGURE 17.45 General view of Tokyo Gate Bridge.

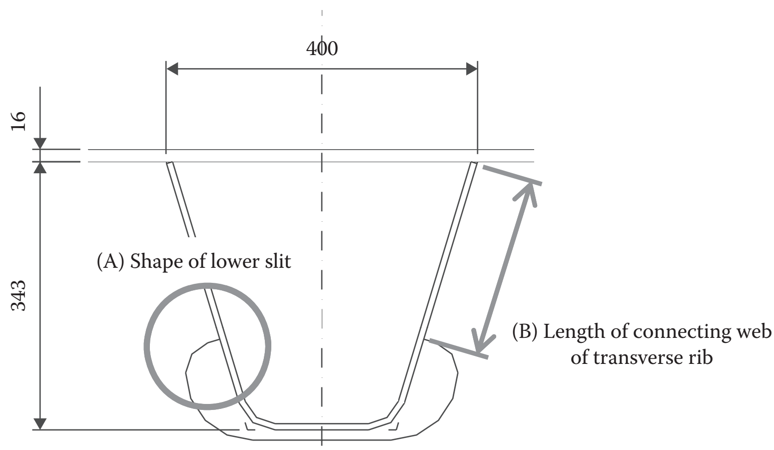


FIGURE 17.46 Basic design of orthotropic steel bridge decks for Tokyo Gate Bridge.

region, the thickness of deck plate is increased from 12 to 16 mm (0.47 to 0.63 in.). When traditional design method for OSD is applied, the change of thickness of deck plate results in 400-mm-wide (15.75-inch-wide) longitudinal closed ribs and 3000–4000 mm (118.11–157.48 in.) intervals of transverse rib.

Figure 17.46 shows the basic design of OSD for Tokyo Gate Bridge. In order to improve fatigue performance of OSD, the shape of lower slit (A), the length of connecting web of transverse rib (B), and other modifications of connection were studied by applying FEM. Truck loading tests and fatigue tests on the full-sized models of OSD were also carried out.

17.5.2 FEM Model

Figure 17.47 is the FEM model of the OSD of Tokyo Gate Bridge. As the first step of analysis, a box section girder model of 120 m long and 24 m wide was established. As the second step of FEM, the submodel of OSD with three longitudinal ribs and three transverse ribs was used. These models consist of shell elements, and the structural hot spot stress (HSS) concept (as shown in Figure 17.48) is applied to evaluate local stresses (Hobbacher 2004).

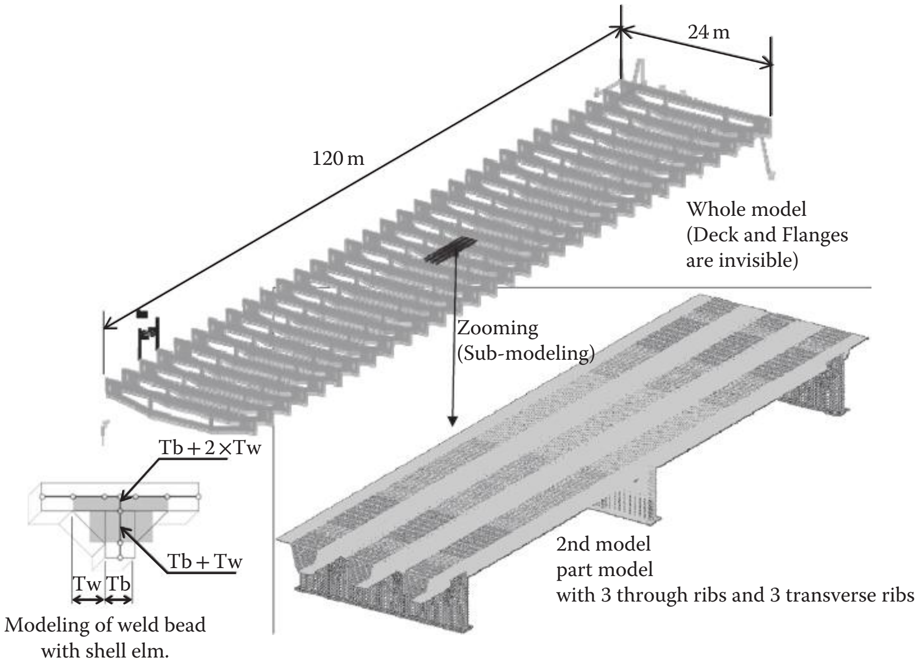


FIGURE 17.47 Finite element method models for study on orthotropic steel bridge decks for Tokyo Gate Bridge.

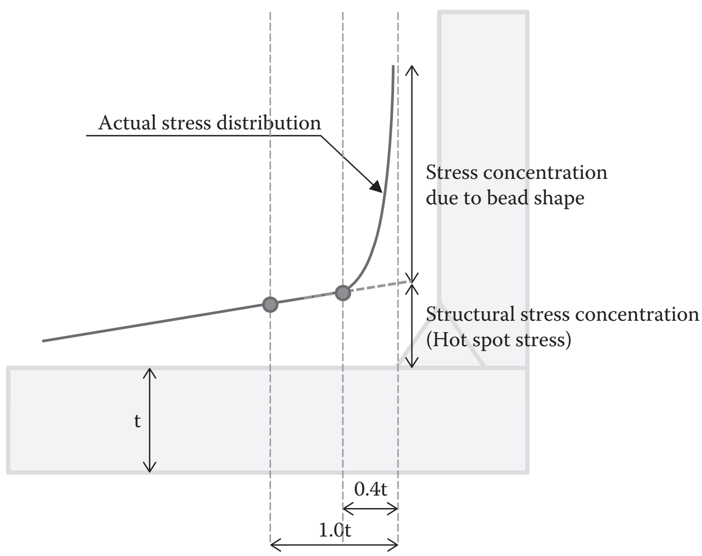


FIGURE 17.48 Structural hot spot concept.

The 80-mm (3.15 in.) layer of asphalt pavements is placed on the plate of OSD. The asphalt pavements also help resist against traffic loads, and the stiffness of asphalt pavements depends on its temperature (Figure 17.28). The elastic modulus of 1500 MPa (217.6 ksi) is applied for analysis. The connection between the center longitudinal rib and center transverse rib is the target of observation.

In order to evaluate stresses at the root of fillet welds which cause FR-, DS-, and DR-type cracks, the third step of FEM was conducted by using solid elements. Effective notch stress (ENS) approach (as shown in Figure 17.49) was applied to evaluate local stresses at the weld root (Suganuma and Miki 2005,

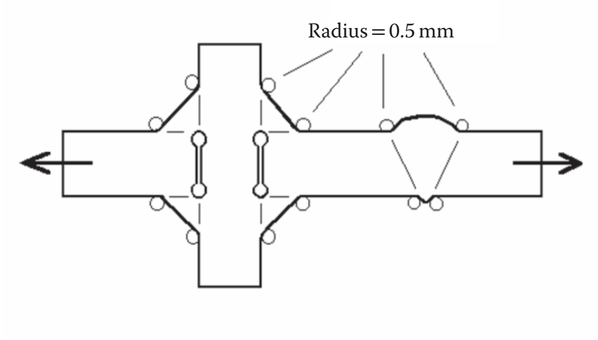


FIGURE 17.49 Effective notch stress concept.

2006 and 2007). Round shape of $R = 0.5$ mm (0.02 in.) is introduced at the tip of weld root and weld toe in this calculation. The tip of weld root and toe is very sharp, in many cases the radius is almost zero, and calculated local stresses at the root were influenced by the size of elements. By applying ENS concept, the dependency on the element size is avoided and direct evaluation of local stresses at different locations, such as weld toe and weld root, becomes possible.

17.5.3 Effects of Loading Positions on Local Stresses

Relationship between loading positions (lanes) and local stresses in the connection between longitudinal closed rib and transverse rib were examined. The load for FEM is shown in Figure 17.50, which is based on the T-20 design load of Japanese Highway Bridge Design Specification. Figure 17.51 shows the loading positions. Lanes A, B, and C are on the center of closed ribs, on the web of the closed rib, and on the center of space between the neighboring closed ribs, respectively.

The traditional slit geometry, as shown in Figure 17.46, was applied for FEM. Figure 17.52 shows the change of HSSs at the toe of the end of slit following the passage of vehicle load on lanes A, B, C, D, and E. The stresses are changed drastically with load lanes. There are two features. One is the relationship between loading lanes and stress. The stress range of lane C is relatively lower than others, although the lane C is right above of the closed rib. The other is about the stress alternation. The stress was tensile when the load lanes are located on the measured side, lanes D and E. On the other hand, the stress was compression when the load lanes are located on the opposite of the measured side, lanes A and B. This means when the loading positions were shifted about 800 mm, stress changed from 80 to −70 MPa (11.6 to −10.2 ksi) and the resulting value of stress range is 150 MPa (20.8 ksi).

The deflection of the closed rib may cause this characteristic feature. The behavior of deflection of the closed rib when the load is located on the center of the span on lane D is shown in Figure 17.53. It is clear that the bottom flange of the closed rib shifts laterally in the free span region. At the cross part of the closed rib and transverse rib with slit form, the deformation of the closed rib was restrained by

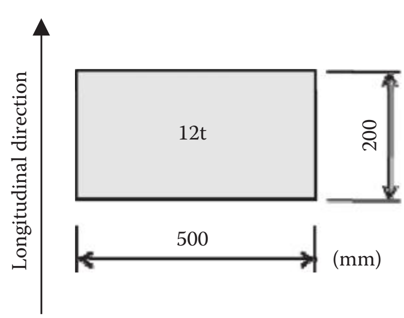


FIGURE 17.50 Load for finite element method.

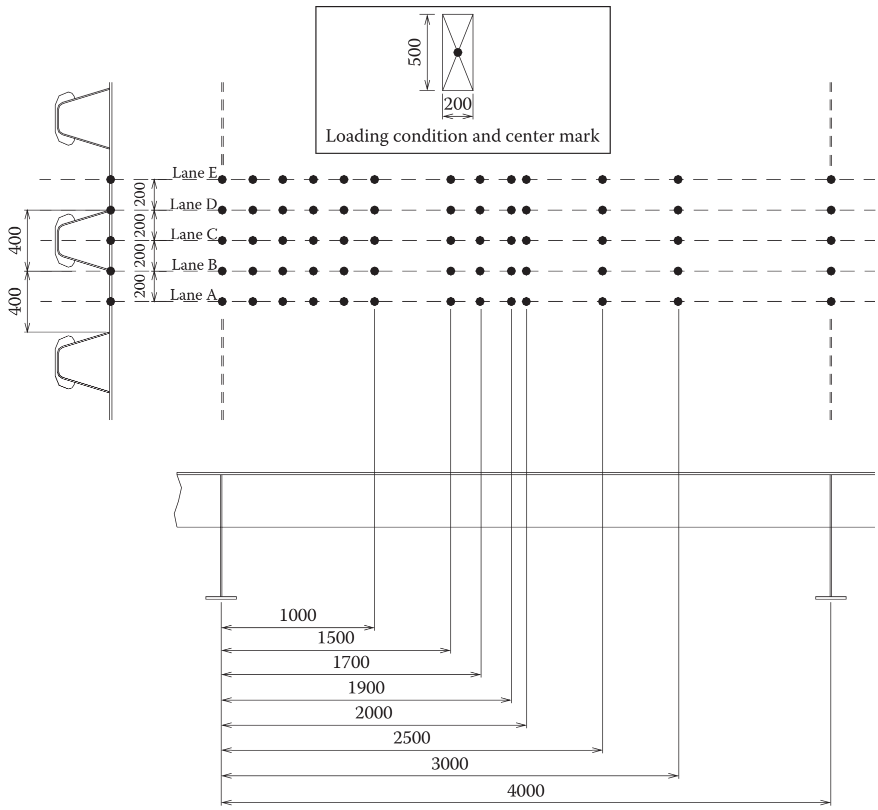


FIGURE 17.51 Loading positions.

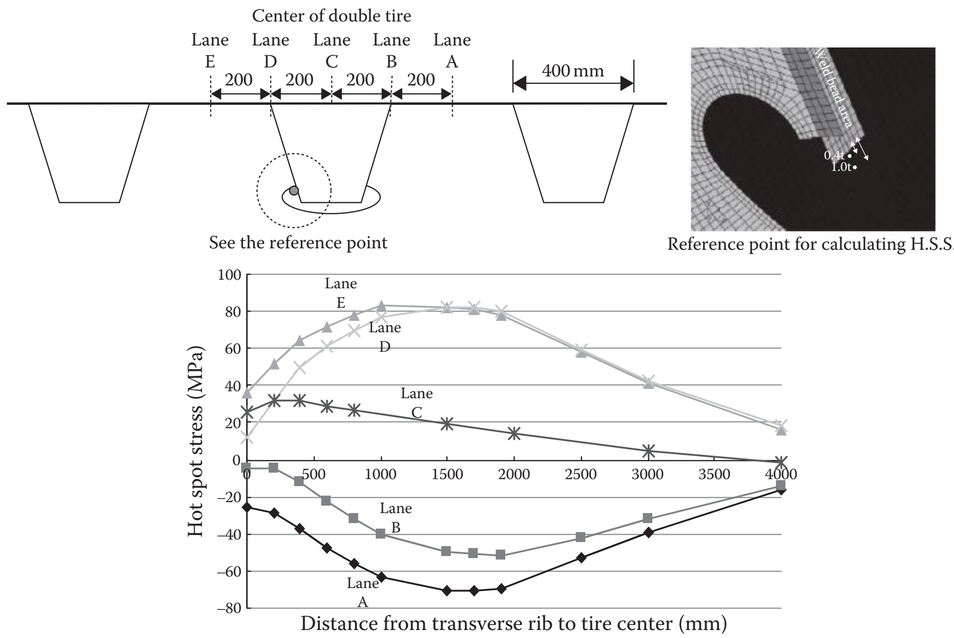


FIGURE 17.52 Change of hot spot stresses with loading position.

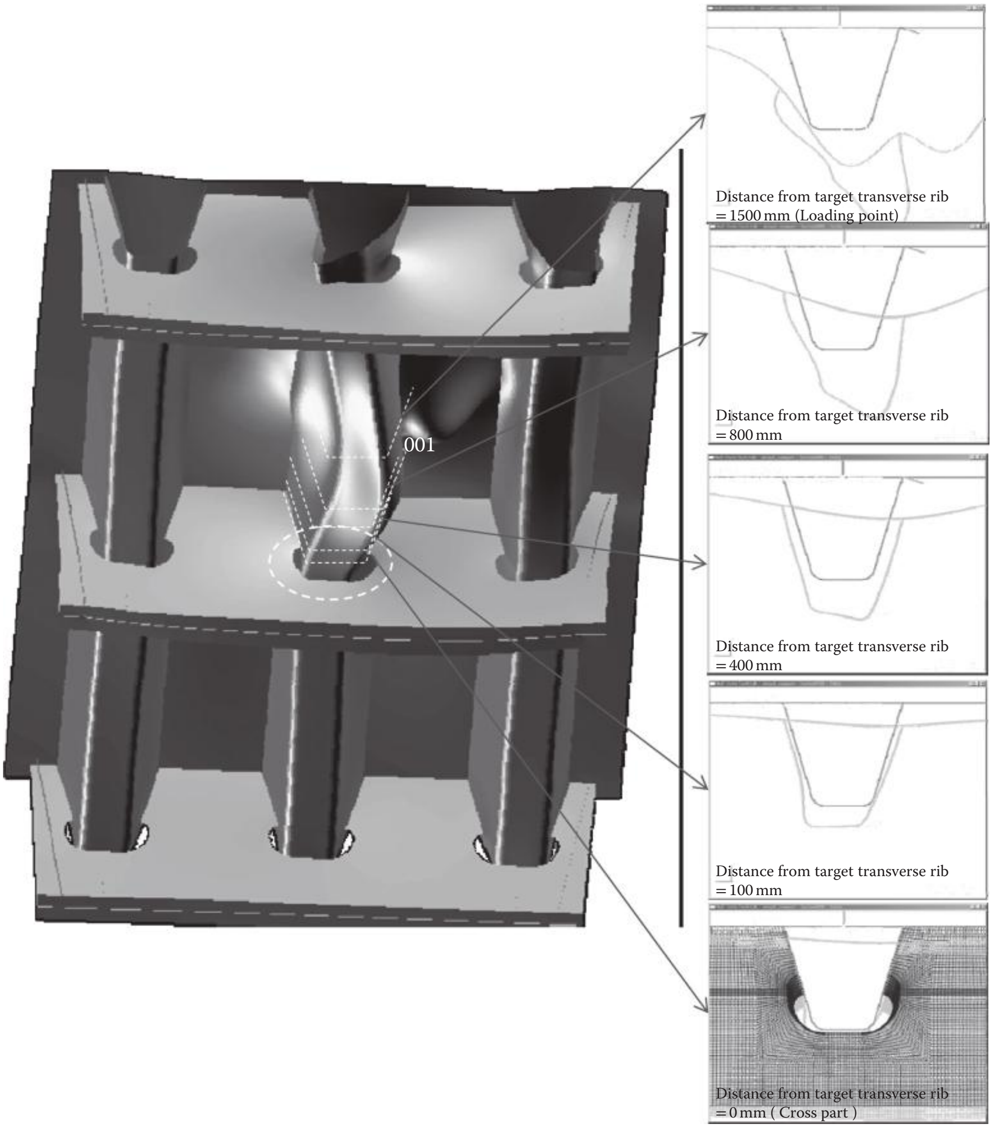


FIGURE 17.53 Deflection of longitudinal ribs because of loading.

the transverse rib. However, the bottom flange of the closed-rib swings because there is slit form. The local stress occurs on the closed-rib side of the slit form boxing weld. Therefore, the load on lane C, right above the closed rib that does not cause distorted deformation, shows lower stress.

17.5.4 Effects of Connection Details

The effects of connection details on local stresses that might cause fatigue damage were mainly discussed by using the behaviors of stresses under lane D loading which generated high stresses.

17.5.4.1 Shape of Slit

Figure 17.54 shows two types of slit, type A (perpendicular shape) and type B (round shape). All connections in both types are fillet welds. The length of connection between the closed rib and the transverse rib is the same in both types. The variations of HSSs on the toe of the closed rib with the

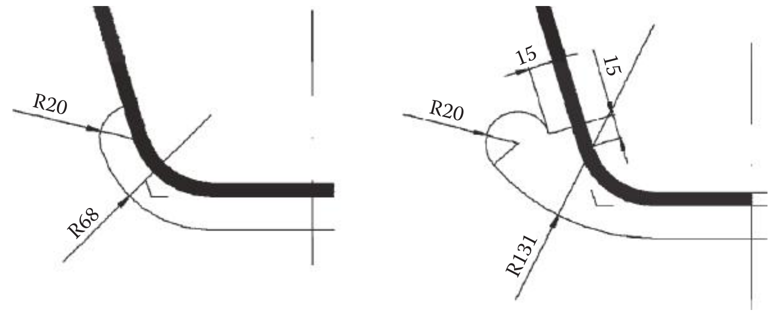


FIGURE 17.54 Two types of slit.

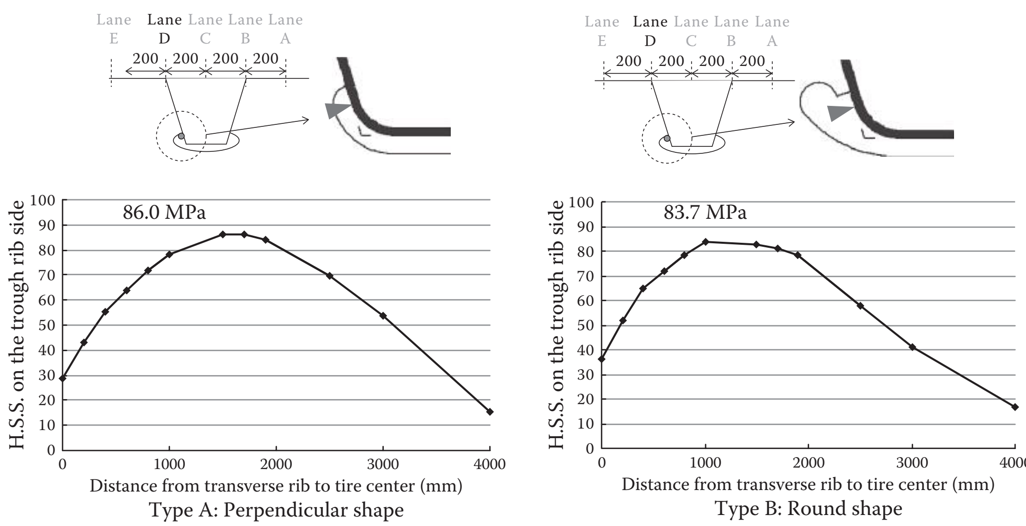


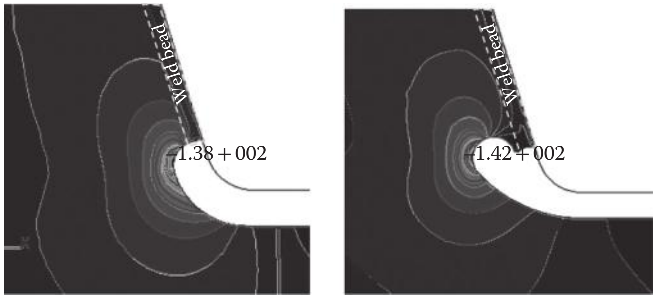
FIGURE 17.55 Variations of hot spot stresses with loading position.

change of loading point are shown in Figure 17.55. The edge of round connection, which behaved like a dumper, gave this small reduction.

The distributions of minimum principal stresses are shown in Figure 17.56, when the load is located on lane D (1500 mm). In type A connection, high stress area expanded to the vicinity of the fillet welds on the transverse rib. On the other hand, the high stress occurs in the region of round edge zone and stress decreases gradually while approaching the toe of fillet welds in the type B connection. These stress distributions indicate type B is more advantageous for fatigue.

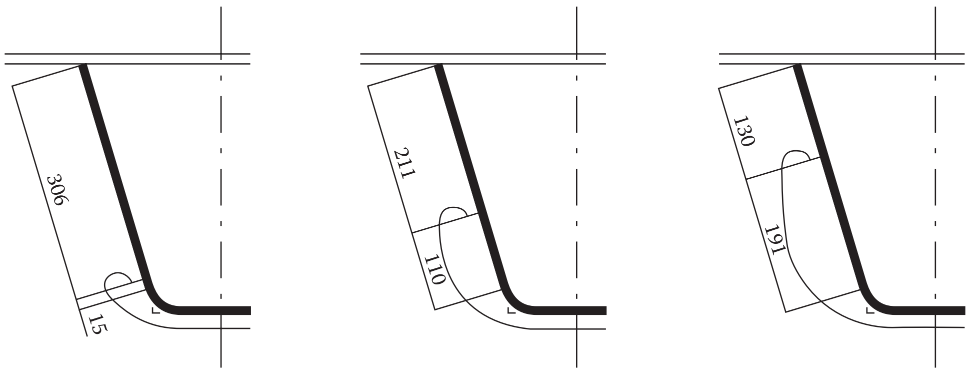
17.5.4.2 Connecting Length between the Web of Transverse Rib and the Closed Rib

In order to examine the effects of the length of connection, three types of the models (Figure 17.57) were prepared. When the traditional design method is applied to assess the shear stress along this connection, the long welded type becomes advantageous. However, the actual stress distribution near this welded connection is completely different from the nominal stress; the decision of structural details based on FEM is more reliable. The slit form is type B of round shape. The length of connection may relate to the restriction of the deflection of the closed rib and may affect the local stresses in the connection zone.



Type A: Perpendicular shape Type B: Round shape

FIGURE 17.56 Distribution of minimum principal stresses near slit.



Long welded length Middle welded length Short welded length

FIGURE 17.57 Three models to examine the effects of length of connection.

The HSSs at the weld toe on the closed-rib side are shown in Figure 17.58. The longest and shortest welded type achieved lower stress on the closed-rib side. In the longest welded type, the restricted small slit form leads to reduction of the local deformation and local stresses. In the shortest welded type, the large slit form gives free deformation around the web of the closed rib. Thus, the HSS on the toe of the closed rib is reduced.

17.5.4.3 Connecting the Bottom Flange of the Closed Rib to Transverse Rib

One way to control the swing mode deflection of the closed rib and to reduce high stresses is to connect the bottom flange of the closed rib to the web of the transverse rib. Two types of connection are considered (Figure 17.59). Type 1 is to connect the short brim of the transverse rib to the bottom flange of the closed rib. Type 2 is to connect all edges of connection. From the viewpoint of welding works, type 1 is easier than type 2.

Various shapes and the lengths of brim are examined. Type 1, shown in Figure 17.59, is the best performance of this connection method. The changes of HSSs are shown in Figure 17.60. The HSS on the closed-rib web side decreases as expected. However, high stress concentration occurred on the surface of the bottom flange. Stress concentration point is changed from the rib web to flange. Figure 17.61 shows stress distribution in the web of transverse rib with type 2 connection detail. However, the highest stress occurs in the corner zones of the next closed rib of the loaded closed rib.

17.5.4.4 Installation of Inner Rib

In order to resist the out-of-plane deformation of web of closed rib at the end of connection, so-called oil canning deformation, inner rib was considered to be installed. Figure 17.62 shows the study models. Parameters are the sizes of the inner rib. Analyses were performed in consideration of stress alternation.

The HSSs on two points are shown in Figure 17.63. The height of an inner rib differs between types A and B. As a result of comparing types A and B, when the stiffness of an inner rib decreases, the stress

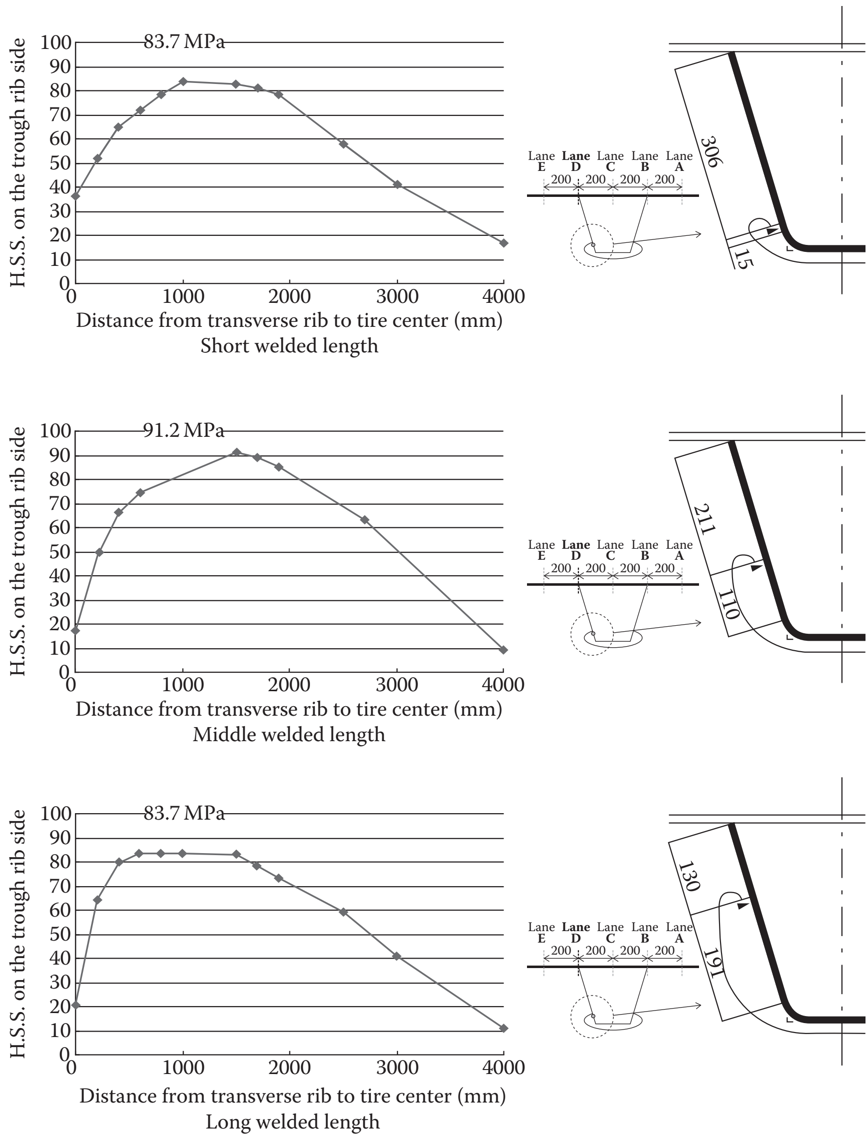


FIGURE 17.58 Variations of hot spot stresses with loading positions.

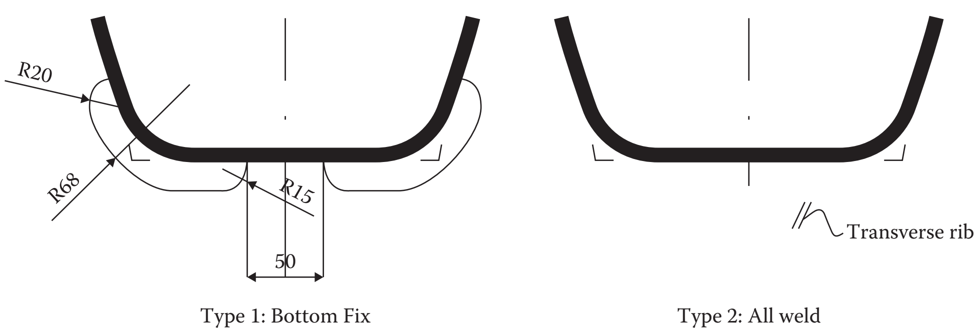


FIGURE 17.59 Modification of connection between longitudinal ribs and transverse ribs.

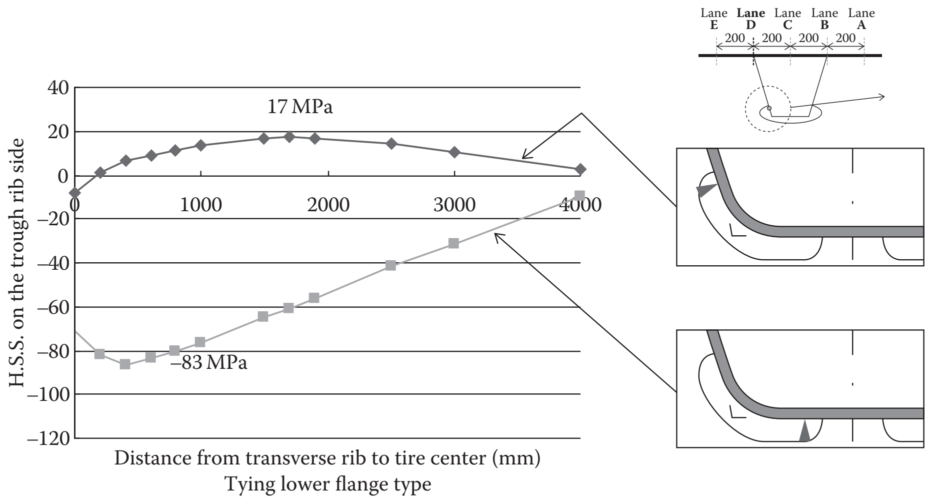


FIGURE 17.60 Variations of hot spot stresses with loading positions (type-1 connection).

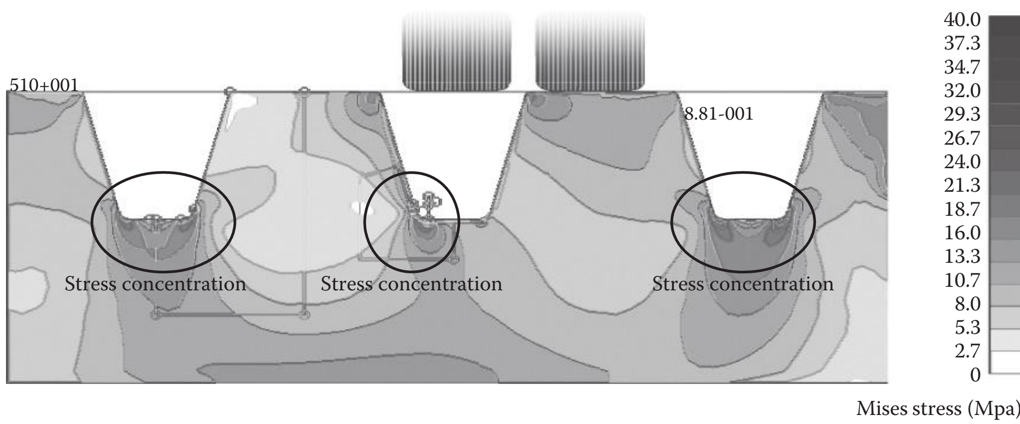


FIGURE 17.61 Minimum principal stress distribution in the connection (type-2 connection).

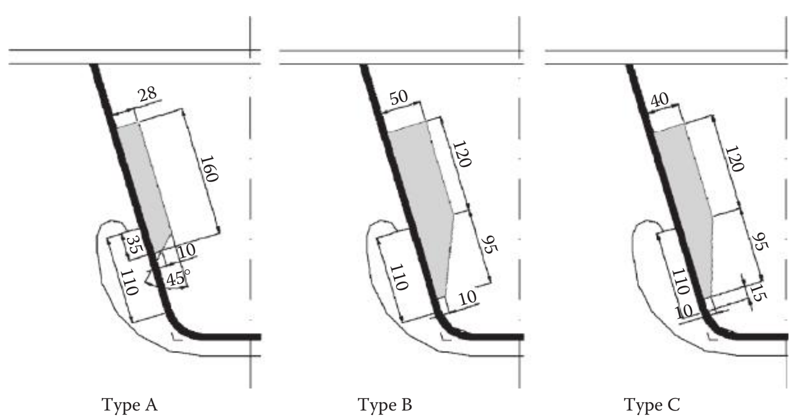


FIGURE 17.62 Models of inner rib.

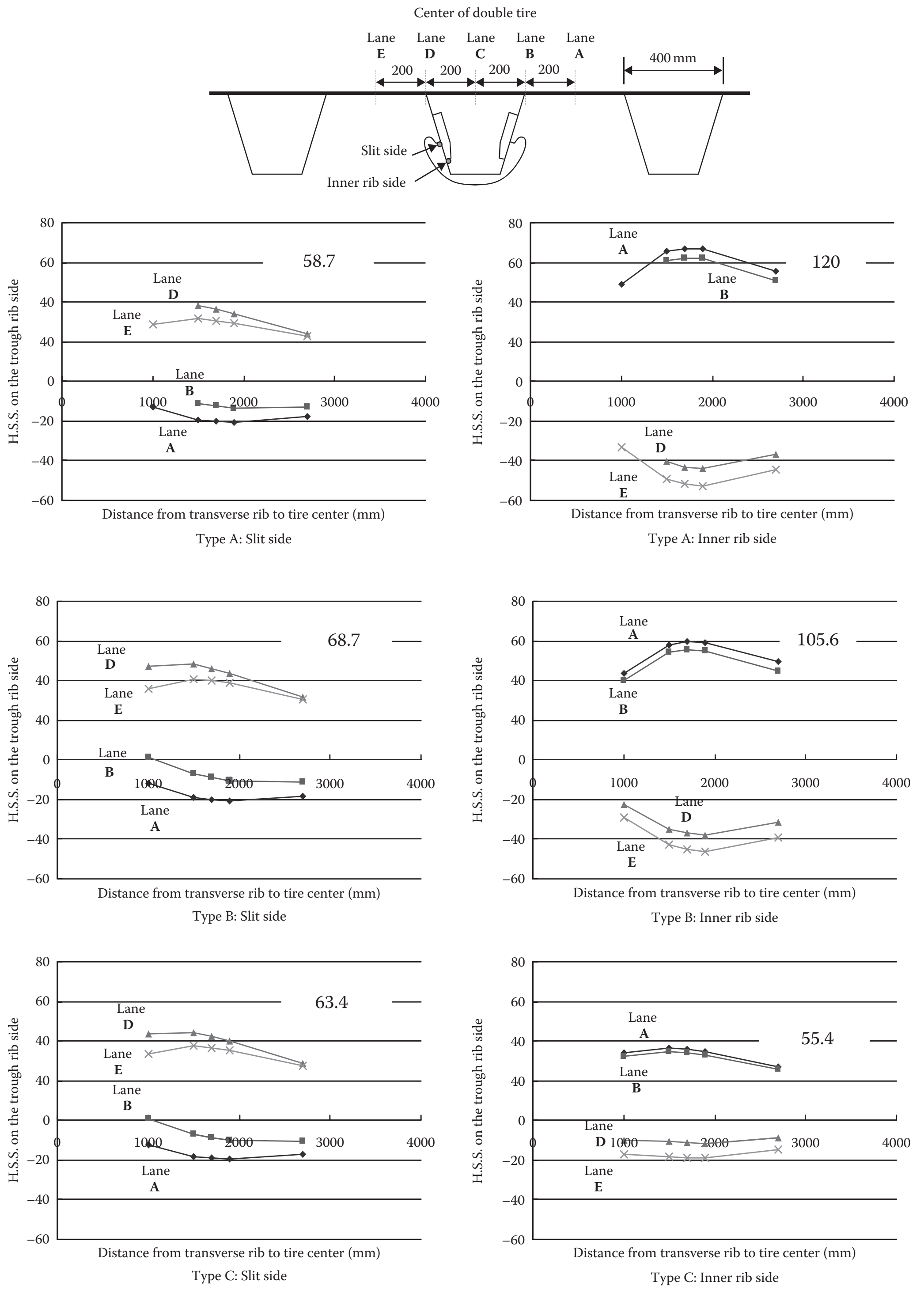


FIGURE 17.63 Variation of hot spot stresses on the web of transverse rib and at the end of inner rib on the web of longitudinal rib with loading position.

range by the side of an inner rib decreases. On the other hand, the stress on the slit side increases. The inner rib was lengthened in order to reduce the stiffness of an inner rib on type C. Although the stress on the inner rib side is decreased further, stress on the slit side is increased only slightly. The long inner rib relaxes the bending of a closed rib.

As a result of parametric study, stress on the slit side can be effectively decreased by increasing stiffness of a slit rear-face part. On the other hand, the stress on the inner rib side can be effectively reduced by lowering the stiffness near an inner rib of bottom side.

17.5.5 Static Loading Tests

17.5.5.1 Specimens

On the basis of the FEM studies, seven types of connection details were designed to evaluate the performances of OSD (Suganuma and Miki 2006 and 2007). All specimens were full scale. Welding procedures were same as for actual structure. These are shown in Figure 17.64. Parameters are as follows:

1. The geometry of the closed ribs
2. The geometry of the transverse rib slit
3. Installation of inner ribs
4. Spacing of transverse ribs

Types A, B, and C were designed to investigate the effects of slit size. The lengths of connection between the transverse rib and the closed rib were 300, 235, and 170 mm (11.81, 9.21, and 6.69 in.). The connection detail of the transverse rib of type A has a perpendicular shape. These specimens have 3 m (9.84 ft) spacing of transverse ribs. Type D was designed to examine the effects of the transverse rib spacing. Type D specimen has 4 m (13.12 ft) spacing of transverse ribs. The bottom cope geometry is the same as type A. Type E specimen was designed to study the effects of connection details same as type B, round shape with boxing welds at the edge (Figure 17.20). The bottom slit size was designed as the smallest that can be constructed with sufficient welding quality. Type F specimen has a narrow width trough rib. The width of trough rib is 330 mm (12.99 in.), which is smaller than others (400 mm [15.75 in.]). The connection detail from transverse rib cutout to closed-rib web is the same as type C, round shape with full penetration and finished by grinding. Inner rib is installed in type F specimen. Type G specimen also has an inner rib. The closed rib width is 400 mm, same as that of types A, B, C, D, and E. The deck thicknesses were set as 16 mm in all the specimens.

Figure 17.65 shows the size and configuration of models consisting of three longitudinal ribs and three transverse ribs same as the FEM model for loading test. The connection detail between the center transverse rib and the center-closed rib is observation area. Static loading tests were carried out by using the loading stage shown in Figure 17.38. The test panel was installed in this stage and loaded by applying a testing truck.

17.5.5.2 Space of Transverse Rib

The effects of space of the transverse rib are examined by comparing type C and type D. Type C and type D specimens have the same connection details: the space of transverse rib is 3000 and 4000 mm (118.11 and 157.48 in.), respectively. The stress on the closed rib was compared.

Figure 17.66 shows the change of stresses when the testing truck moved on the deck from one end to the other end of the specimen. The center of double tires coincides with the web of closed rib. Measuring point is on the web of the closed rib in the connection, 10 mm apart from the tip of boxing welds. The horizontal axis shows the relative loading position to specimen length. This stress component is caused by out-of-plane displacement of the web of the closed rib, which is an oil canning deformation. The stress becomes higher when the space of transverse rib is longer. The largest stresses are 18.5 and 26 MPa (2.7 and 3.8 ksi) correspondingly. The ratio of stress increase is 1.4. It is almost the same as that of spacing increase, which is 1.3.

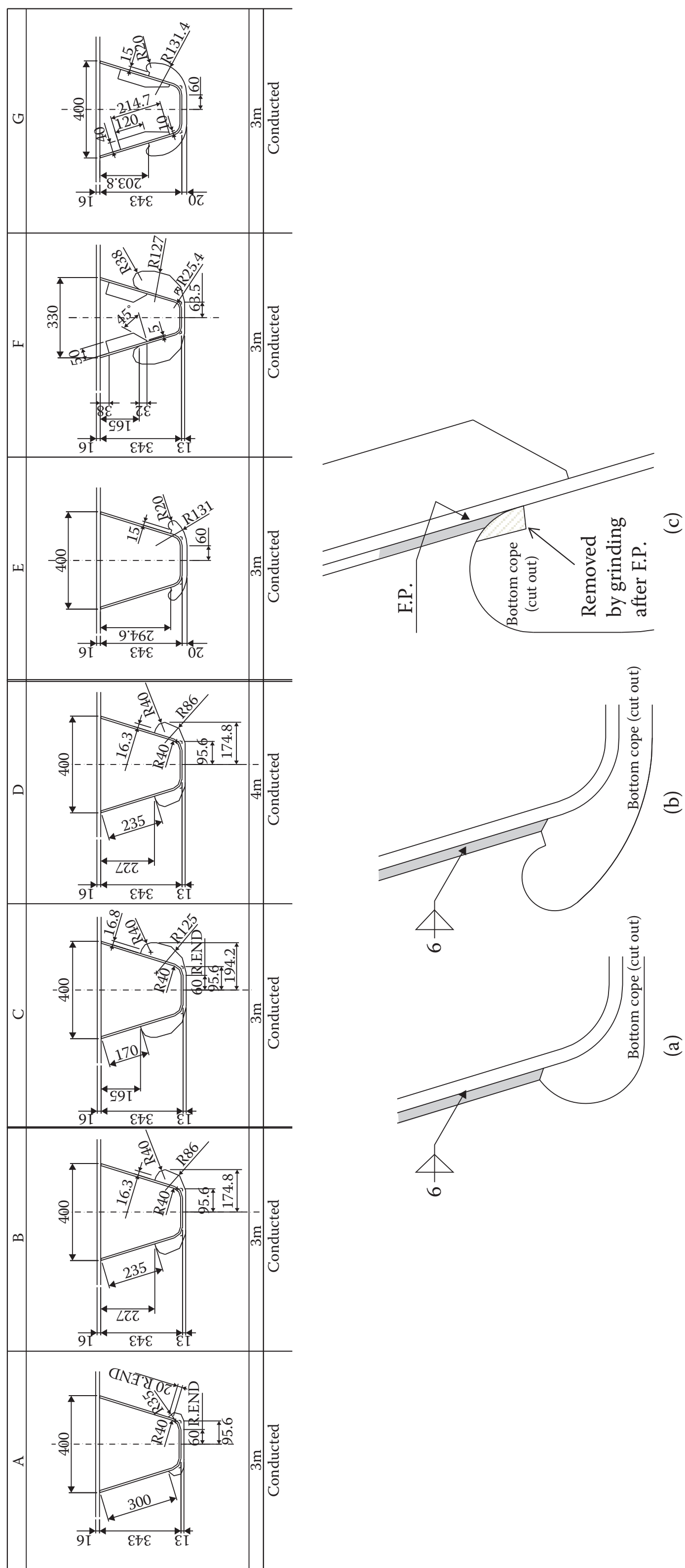


FIGURE 17.64 Connection details of specimens for static loading tests: (a) Perpendicular type; (b) Round type (butt weld); (c) Round type (full penetration).

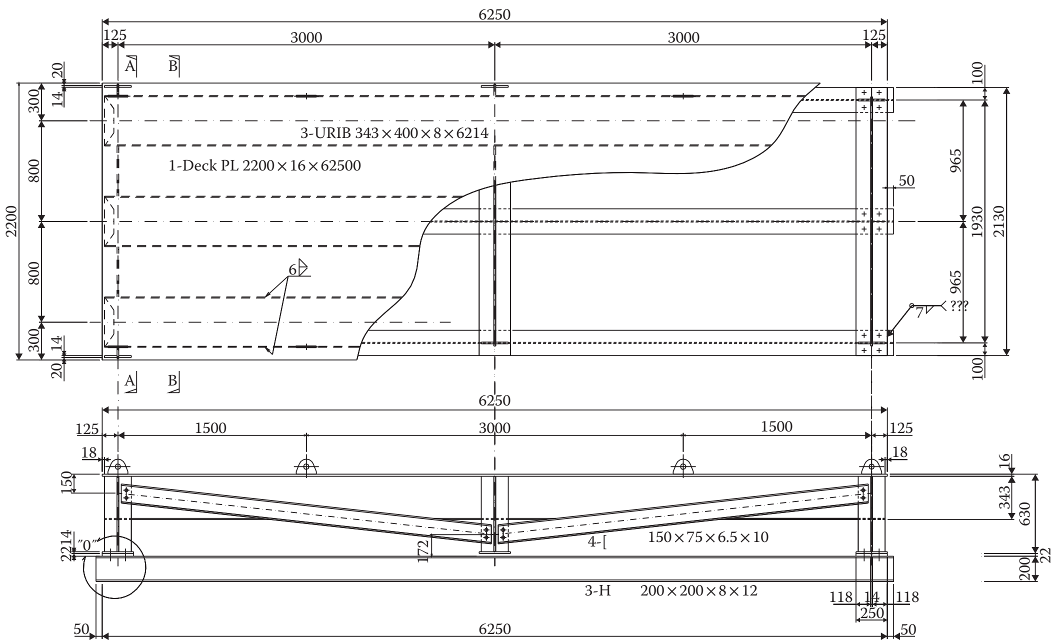


FIGURE 17.65 Size and configuration of specimen.

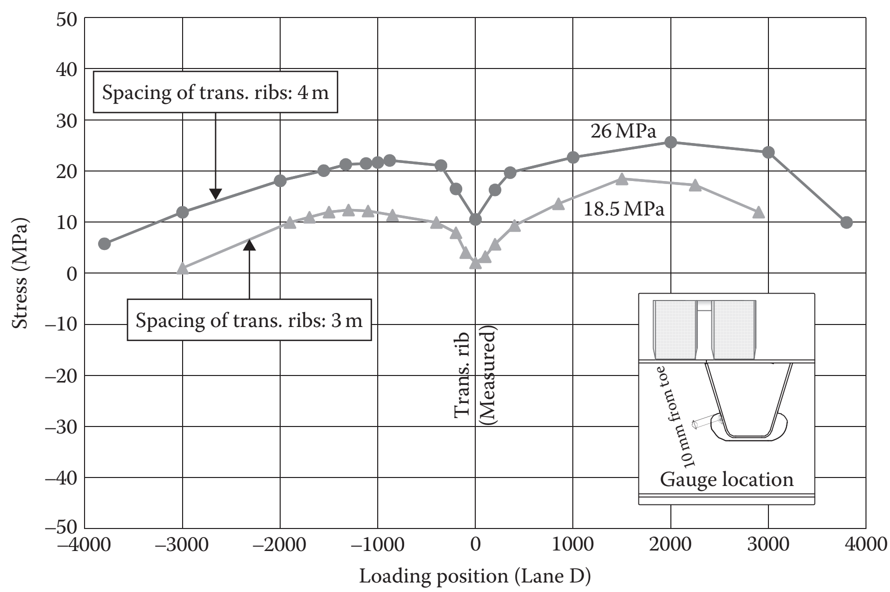


FIGURE 17.66 Variation of stresses with loading positions, effects of the space of transverse rib.

17.5.5.3 Shape of Slit

17.5.5.3.1 Stresses on the Closed Rib

The loading position and the measurement point in each specimen are shown in Figure 17.67. The measuring point is 10 mm (0.39 in.) apart from the toe of boxing welds in types A, B, C, and F. However, they are 8 and 3.2 mm (0.32 and 0.126 in.) apart from boxing weld toe in types E and G. In case of type G, the gages were put inside the closed rib. All measurement values are converted into per 10 tonf unless mentioned in this chapter.

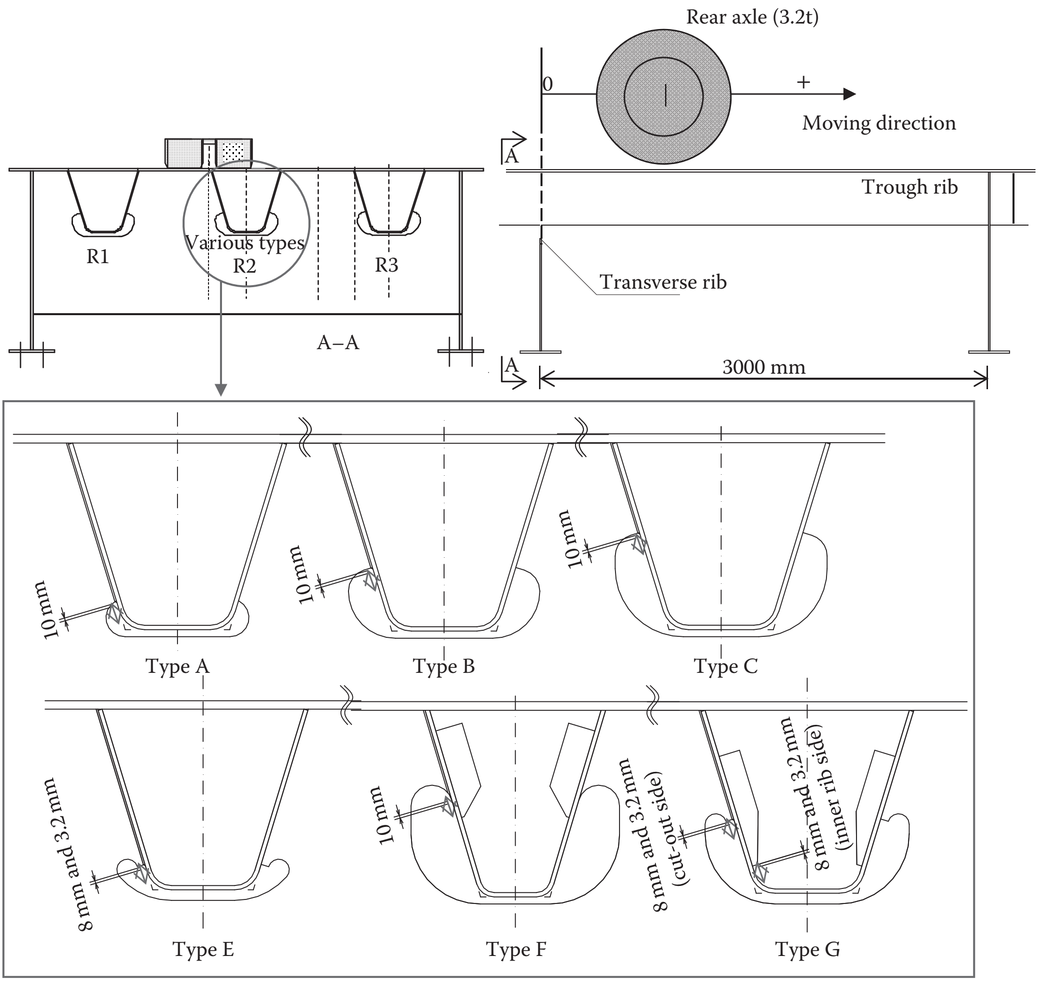


FIGURE 17.67 Stress-measuring positions (web of longitudinal rib).

The HSSs of types A, B, C, E, F, and G are shown in Figure 17.68. The differences of stresses between types A, B, and C specimens are very small, which is the same as the results of FEM (as shown in Section 17.5.5). Comparing the results of types A and E, the maximum stress is almost same; however, the location of load which provides the peak stress shifts to the span center. Although the sizes of slit of types A and E are almost same, the slot shapes are different, one rectangular shape and another round shape.

The stresses in type F and G connections are smaller than others that do not have inner ribs. The stress ranges of types F and G are almost same. The measured results on type G specimen indicated that stresses near the toe of the inner rib are relatively low compared with those on the toe of the slit side.

17.5.5.3.2 Stresses on the Transverse Rib Side

The loading position and the measurement point in each specimen are shown in Figure 17.69. The measurement point is 10 mm (0.39 in.) from weld toe and edge of cutout in types A, B, C, and F.

Figure 17.70 shows the stresses from types A, B, C, and F slit form, caused by out-of-plane bending of transverse rib. These stresses are minimum principal stresses. The long connection of the closed rib

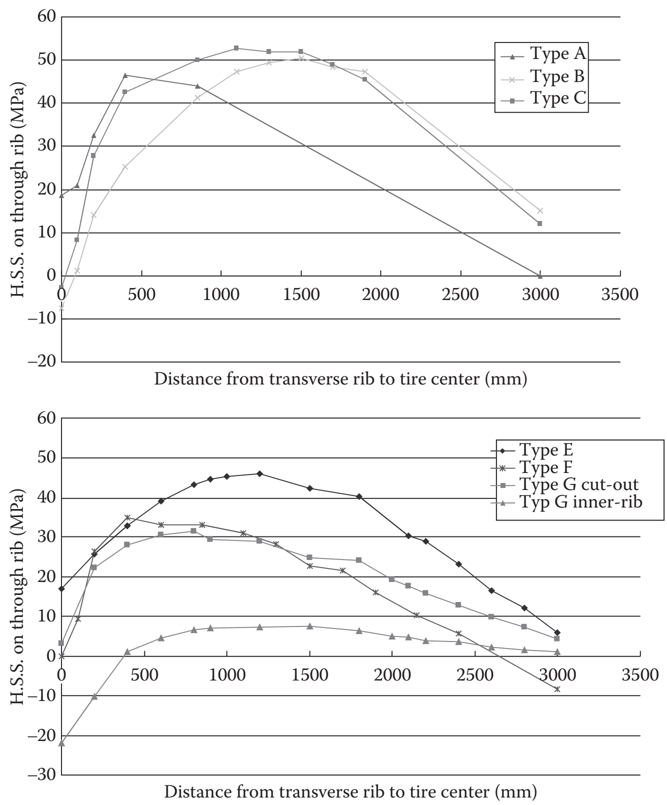


FIGURE 17.68 Variation of hot spot stresses with loading position.

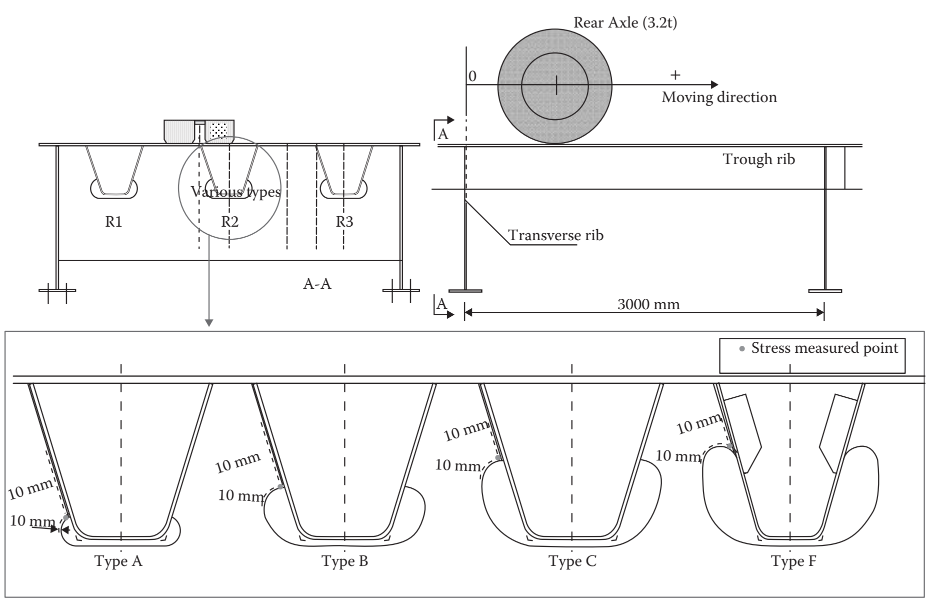


FIGURE 17.69 Stress-measuring positions (web of transverse rib).

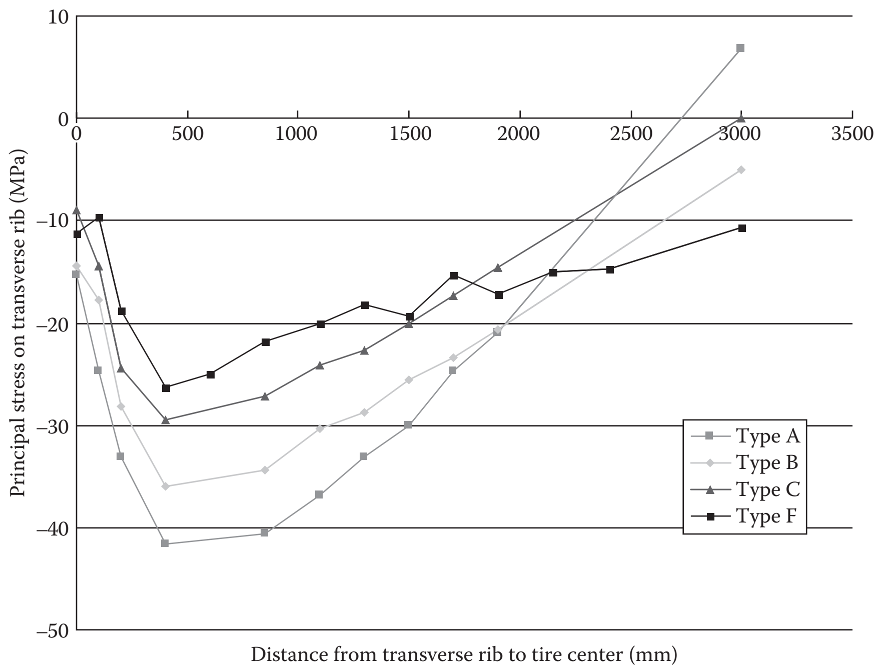


FIGURE 17.70 Variation of hot spot stresses with loading position.

and the transverse rib resulted in severe out-of-plane bending on the transverse rib. Thus, type A with the longest connection shows the highest stress, and the type C with the shortest connection shows the smallest stress. Type F shows the best performance in these four types. Since type F connection has a round-shaped connection detail, the stress concentration would have moved from near the connection area, which agrees well with results from FEM.

17.5.5.4 Connection of Deck Plate and Closed Rib

The effects of the width of the closed rib of 320 and 400 mm (12.60 and 15.75 in.) and the sizes of tire on the out-of-plane bending stresses in the deck plate are shown in Figure 17.71. The stress measurements were performed by using a single tire and a double tire. The width of single tire is narrower than the closed-rib width and the width of double tire is wider than the closed rib width. The weight of single tire is 14 kN (2.70 kip) and the weight of double tire in this test is 32 kN (7.19 kN). The out-of-plane stress components are calculated by using the measured stresses on surface and back surface.

Figure 17.72 shows the out-of-plane stress at 40 mm apart from the weld of the closed rib on deck plate corresponding to the truck-load locations in the transverse direction. When a single tire is loaded on the web of trough rib, very high out-of-plane bending stresses are introduced in the deck plate, which are higher than those caused by double-tire loading, although the weight of single tire is much smaller than that of double tire. The load with single tire gave more severe stress. In the case of double-tire loading, the center loading causes peak stress.

In the cases of single tire or double tire, the connection of type F shows relatively lower out-of-plane stresses. The closed rib width of type F that is 330 mm, others have 400 mm, causes lower stresses because the closed web behave like a soft support of deck plate.

While comparing specimens A, B, and C with the same closed rib width, we get the same result. This means that differences of the bottom slit geometry have little effect on the deformation of deck plate. Thus, closed rib width and deck thickness govern the deformation of deck plate.

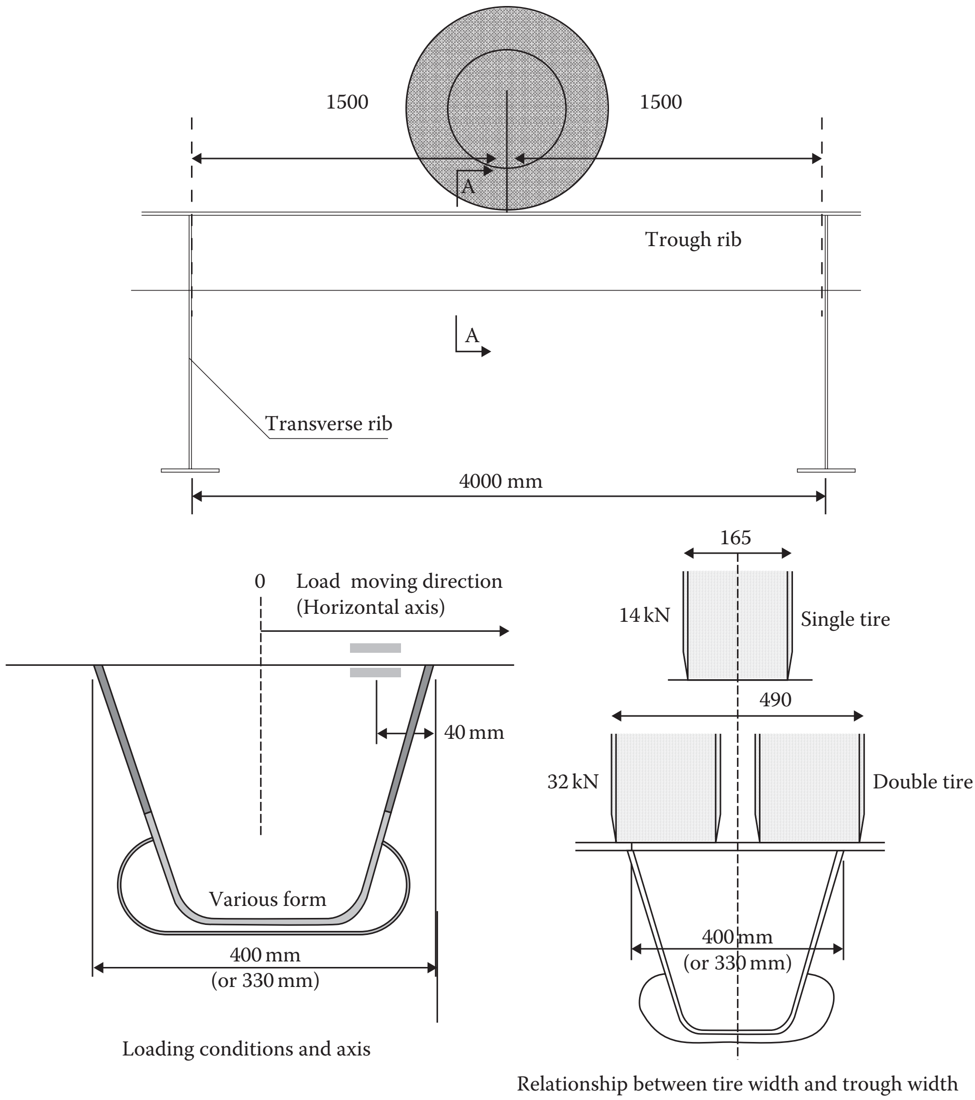


FIGURE 17.71 Stress-measuring points on both surfaces of deck plate and loading condition.

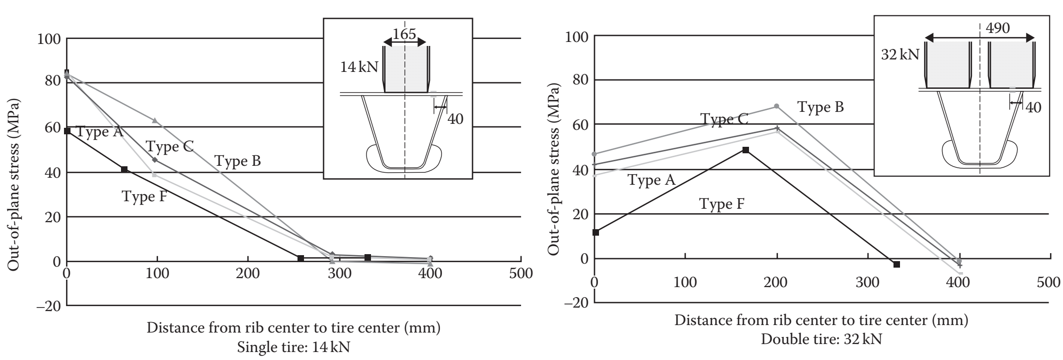


FIGURE 17.72 Change of out-of-plane stress components with loading position.

17.5.6 Fatigue Test

17.5.6.1 Specimens

Fatigue test were conducted on specimen with four slit forms. They are types B, E, F, and G. Type B was chosen because of proper balance of stress on the closed-rib side and transverse rib side at the connection. Type E was chosen because of its good performance without attaching additional components like inner ribs. Types F and G were chosen because of inner ribs and smooth connection detail at the cutout portion. Type G is predominant in profitability and workability if it has as much the same fatigue resistance as that of type F. This is because type G has a wider closed rib width and uses butt weld instead of full penetration. All specimens consist of three transverse ribs. The target of fatigue test is the connection at the center of specimen.

17.5.6.2 Fatigue Test Setup

In order to simulate the truck-moving load, the three-actuator system was applied to the fatigue load test. As shown in Figure 17.73, the center actuator is loaded right above the center transverse rib in longitudinal direction and the other two actuators are arranged at 600 mm (23.62 in.) from the

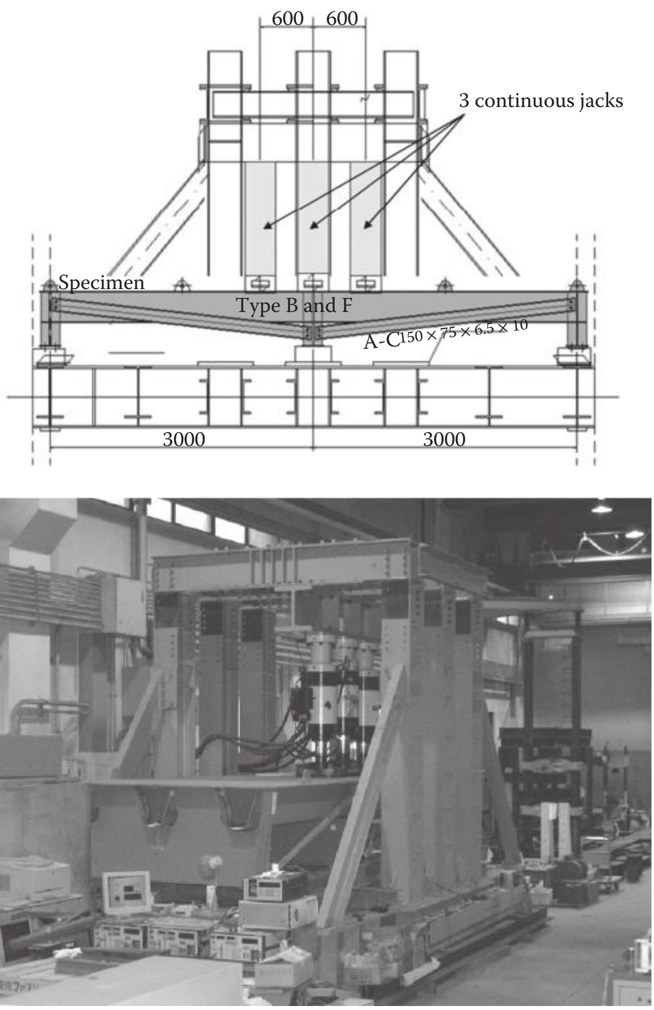


FIGURE 17.73 Three-jack system to simulate moving loads.

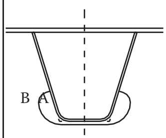
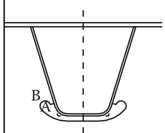
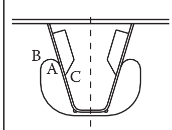
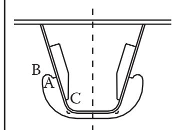
Type			Type C	Type E	Type F	Type G
Shape of rib and bottom cope	Shape of rib and bottom cope					
Jack load			2t - 27t	2t - 22t	2t - 27t	2t - 22t
Stress range (MPa)	End of cut-out on trough rib (H.S.S.)	A	177	108	67	33
	Edge of cut-out on trans. rib	B	108	152	110	113
	Tip of inner rib (H.S.S.)	C			75	20

FIGURE 17.74 Connection details for fatigue tests.

center actuator. This distance of 600 mm (23.62 in.) corresponds to the loading position that generates high stresses in the connection (Figure 17.27). These three actuators are controlled by computer to simulate the variation of stresses under moving vehicles. One series of three continuous actuators loading gives one cycle of stress frequency. The loading frequency was 1.0 Hz. The stress ranges in the specimens are given in Figure 17.74.

17.5.6.3 Fatigue Test Results

17.5.6.3.1 Type B Connection

Cracks were detected at 4×10^5 cycles because the gage on the tip of boxing weld changed drastically. The fatigue cracks were observed by using magnetic particle method. As a result, seven cracks were found in the specimen (Figure 17.75). The crack propagated from the boxing weld to the web of the closed rib. An out-of-plane bending of the closed rib caused the cracks.

17.5.6.3.2 Type E Connection

A crack was found at 2×10^6 cycles. The fatigue cracks were observed by using magnetic particle method. The crack length on the surface was 8 mm (Figure 17.76). It propagated from the boxing weld to the web of the closed rib. An out-of-plane bending of the closed rib caused the cracks. The fatigue life of this point was predicted to 1.6×10^6 cycles from static load test. Therefore, it can be said that it was a reasonable result.

17.5.6.3.3 Type F and Type G Connections

Cracks were not detected until 2×10^6 cycles. It was confirmed that types F and G were the structures with high fatigue resistance. Since inner rib worked effectively, stress concentration was relaxed due to smaller out-of-plane bending of the closed rib.

Comparing types F and G, type F has 330 mm (12.99 in.) closed rib width and type G has 400 mm (15.75 in.) closed rib width. Therefore, type G has an advantage when the same area of plate was stiffened. On the other hand, type F was welded by full penetration between the closed rib and the transverse rib, and was finished with grinding. Type G was welded by butt weld. From the viewpoint of workability, type G is clearly advantageous.

17.5.6.3.4 Fatigue Strengths of These Details

Figure 17.77 shows the fatigue test results using the ranges of hot spot stress and compared with fatigue design curves of the International Institute of Welding fatigue design recommendations (Hobbacher 2004).

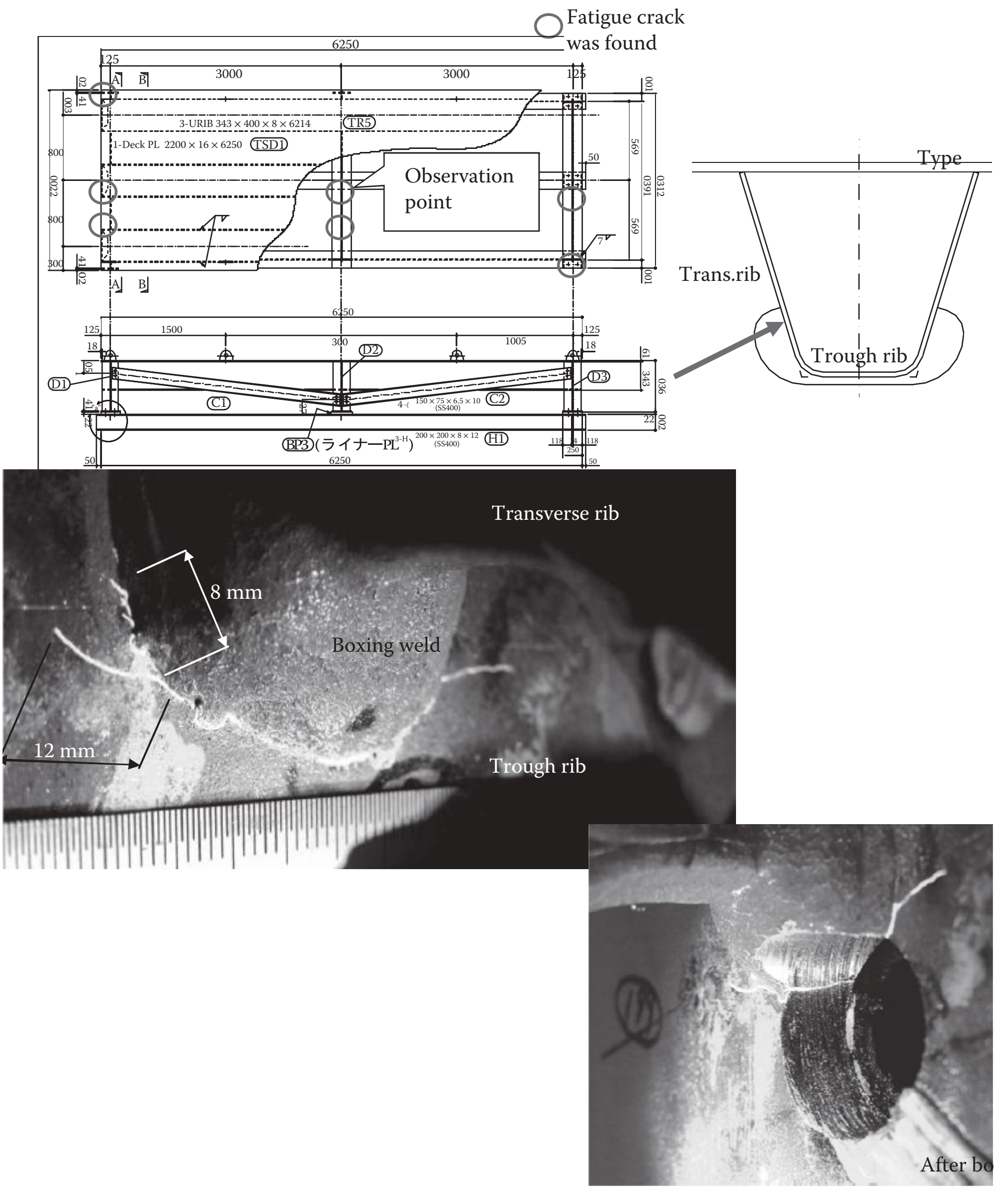


FIGURE 17.75 Fatigue cracks in the type B connection.

17.5.7 Details of Orthotropic Steel Decks for Tokyo Gate Bridge

Figure 17.78 shows the details of OSD for Tokyo Gate Bridge as the results of these studies. The main features are as follows:

1. Thickness of deck plate: 16 mm (0.63 in.)
2. Size of closed rib: 400 mm (15.75 in.) wide, 343 mm (13.50 in.) high, 8 mm (0.63 in.) thick
3. Space of transverse rib: 4 m (13.12 ft)
4. Shape of slit: round
5. Length of connection: short
6. Inner rib: longer rib with reducing width in bottom part

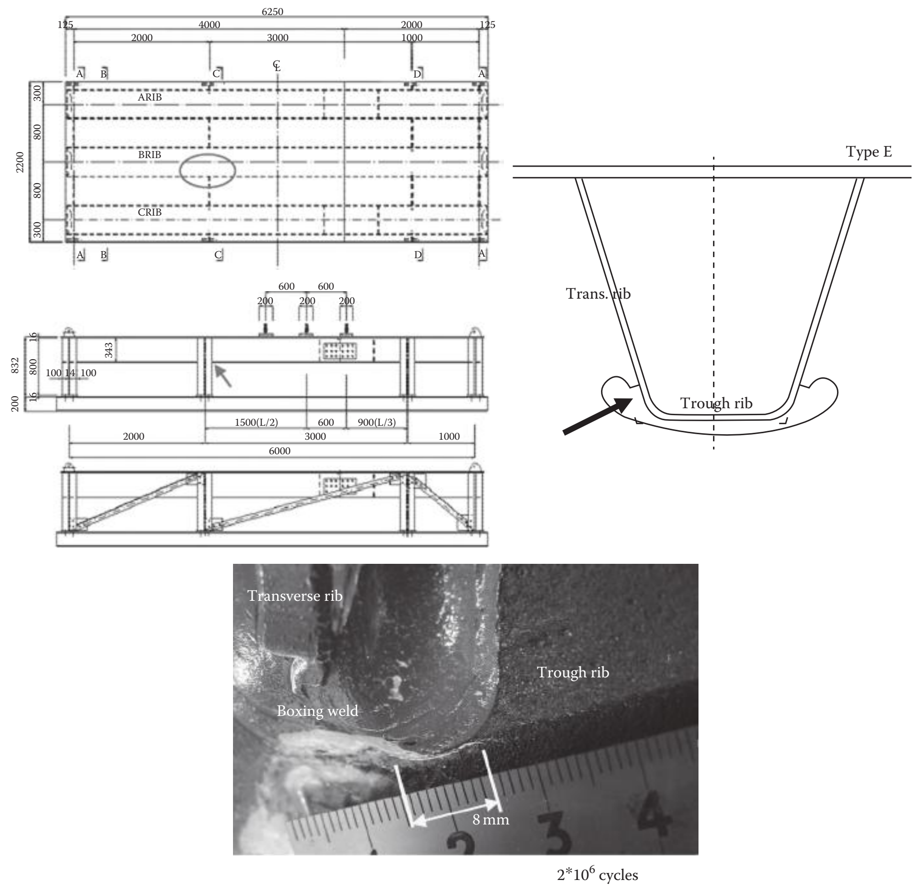


FIGURE 17.76 Fatigue crack in the type E connection.

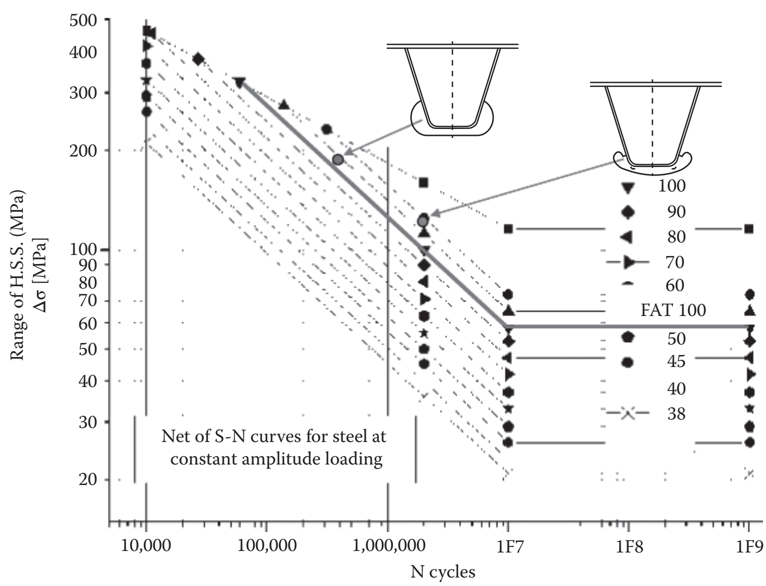


FIGURE 17.77 S–N arrangement of fatigue tests by applying hot spot stress concept.

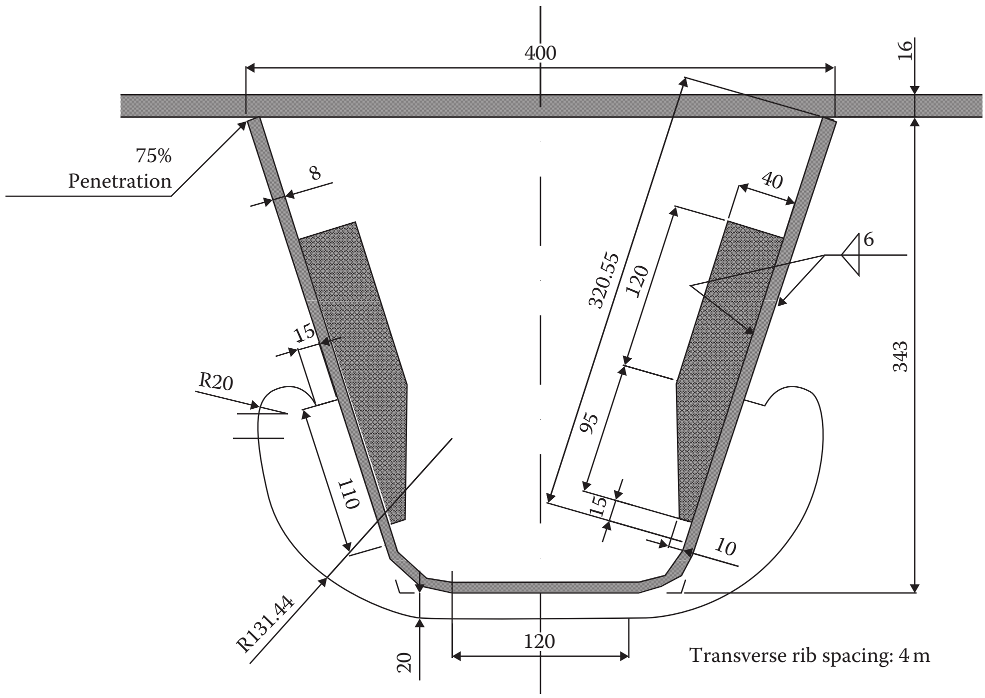


FIGURE 17.78 Connection details of orthotropic steel bridge decks for Tokyo Gate Bridge.

17.6 Summary

Main causes of fatigue damage and corresponding strengthening procedures are as follows:

1. Weld defects remained in the butt welds of closed ribs with backing plate
    a. Quality control in fabrication, such as appropriate width of groove and tight installation of backing plates
2. Complicated deflections of plate elements due to loads such as out-of-plane deflections in deck plates, longitudinal ribs, and transverse ribs
    a. Increase stiffness of deck plates and improve the connection details between longitudinal ribs and transverse ribs
3. High stress concentration along the root of longitudinal welds between deck plates and closed ribs due to the out-of-plane deflection of deck plates
    a. Decrease out of deflection of deck plate. Adequate penetration, change to full penetration or both side welds

References

1. Gurney, T. R. 1992. “Fatigue of steel bridge decks,” *Issue 8 of State-of-the-art review / Transport Research Laboratory*, H.M. Stationery Office, London, UK.
2. Hobbacher, A. 2004. “IIW Recommendations for fatigue design of welded joints and components, IIW-XIII-1965-03/XV-1127-03,” International Institute of Welding, *WRC Bulletin 520*, Welding Research Council, New York, NY.

3. Miki, C. 2006. "Fatigue damage in orthotropic steel bridge decks and retrofit works," *International Journal of Steel Structures*, 6(2), 255–267.
4. Miki, C. and Konishi, T. 2009. "Inspection and retrofitting of fatigue damaged orthotropic steel deck," *Proc. of 2nd International Conference on Fatigue and Fracture in the Infrastructure*, Philadelphia, PA, July 26–29. Lehigh University, Bethlehem, PA.
5. Miki, C., Suganuma, H., Tomizawa, M., and Machida. F. 2005. "Cause study on fatigue damage in orthotropic steel bridge deck." *Proceedings of JSCE*, 780, 57–69, (In Japanese).
6. Miki, C., Suzuki, K., Sasaki, E., Ishida, M., and Takamori, H. 2007. "Preventive works for fatigue damage in orthotropic steel bridge deck by SFRC pavement and long term monitoring of the composite action," *Doboku Gakkai Ronbunshuu A,* 62(4), 950–963, (In Japanese).
7. Ono, S., Shimozato, T., Masui, T., Machida, F., and Miki, C. 2005. "Retrofitting method for existing orthotropic steel deck," *Proceedings of JSCE,* 801, 213–226, (In Japanese).
8. Suganuma, H. and Miki, C. 2005. "Investigation of fatigue crack on the weld of deck plate and trough rib with effective notch stress method," *IIW-XIII-2084-05*, International Institute of Welding, Villepinte, France.
9. Suganuma, H. and Miki, C. 2006. "Investigation of the high fatigue resisted slit form on the cross of trough rib and transverse rib on the orthotropic steel deck," *IIW-XIII-2121-06*, International Institute of Welding, Villepinte, France.
10. Suganuma, H. and Miki, C. 2007. "Full size fatigue tests of the new orthotropic steel deck system," *IIW-XIII-2164-07*, International Institute of Welding, Villepinte, France.
11. Suganuma, H., Miki, C., and Tomizawa, M. 2003. "Identification of the cause of fatigue damage in an orthotropic steel bridge deck structure with box girder," *IIW-XIII-1973–03*, International Institute of Welding, Villepinte, France.

18

Life-Cycle Performance Analysis and Optimization

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18.1 Introduction

Improving the condition and safety of deteriorating bridges usually requires large amount of investments. According to ASCE (2009), more than 26% of the bridges in the United States are structurally deficient, and $17 billion (in 2006 dollars) annual investments are is required over the next 50 years to eliminate all existing and arising bridge deficiencies over this time interval. Bridge managers are usually faced with the problem of allocating limited financial resources in a cost-effective manner to maintain adequate functionality of deteriorating bridges (Liu and Fran gopol 2005a). Life-cycle analysis has been well recognized as a significant tool to maximize the cost-effectiveness of improving bridge condition and safety in addition to extending service life. Therefore, understanding of life-cycle analysis is necessary and essential for bridge engineers and managers.

This chapter presents general concepts of bridge life-cycle performance analysis and optimization. To understand the structural performance prediction for life-cycle analysis under uncertainty, critical deterioration mechanisms for bridges, such as corrosion and fatigue, and reliability concepts are discussed. Furthermore, effects of inspection, monitoring, and maintenance on bridge life-cycle performance are presented. Considering these effects, several approaches for life-cycle optimization including optimum inspection and maintenance planning are introduced. General concepts of optimal management using multiobjective optimization and their applications are also provided.

18.2 Life-Cycle Performance Analysis

The performance of a bridge structure undergoes gradual deterioration due to various environmental and mechanical stressors. To ensure bridge safety during its service life, maintenance and risk mitigation are required in a cost-effective way (Frangopol et al. 2001). On the basis of life-cycle analysis, limited financial resources should be allocated so that the bridge performance can be improved efficiently (Das 1999).

18.2.1 Life-Cycle Concept

Life-cycle analysis of deteriorating bridges is a systemic method to evaluate the effects of time-dependent deterioration processes, loading conditions, and maintenance actions on the structural performance (Frangopol and Liu 2006). Figure 18.1 shows the effect of maintenance actions on time-dependent bridge performance and maintenance cost. As shown in Figure 18.1a, the bridge performance P, with an initial value P_o, decreases after the time of damage initiation t_{ini} due to the deterioration processes and loadings. If maintenance actions are applied at times $t_{main,1}$ and $t_{main,2}$, the bridge performance can be improved by ΔP, the bridge deterioration can be delayed by t_d, and the deterioration rate can be reduced. The maintenance costs $C_{M,1}$ and $C_{M,2}$ are required at time $t_{main,1}$ and $t_{main,2}$, respectively, as shown in Figure 18.1b. The total maintenance cost is $C_{M,1} + C_{M,2}$. Using life-cycle cost analysis and optimization, the times of maintenance application and types of maintenance can be determined.

One of the most critical steps for life-cycle analysis is to assess and predict the structural performance of a deteriorating bridge. Generally, the structural performance prediction includes high uncertainties (Frangopol 2011). For example, the initial bridge performance P_o, time of deterioration initiation t_{ini}, bridge performance improvement ΔP, and deterioration delay time t_d in Figure 18.1a are uncertain. To treat uncertainties associated with the structural performance prediction rationally, reliability-based life-cycle analysis was introduced and investigated by Frangopol et al. (1997a,b), Estes and Frangopol (1999), Kong and Frangopol (2003, 2005), Ang and De Leon (2005), and Moan (2005), among others.

Through a single- or multiobjective optimization, the reliability-based life-cycle analysis can provide the (1) optimum expected total cost including the initial, inspection, maintenance and repair costs, in addition to the cost associated with structural failure during a predefined lifetime; (2) optimum times and types of inspection, maintenance, and repair; (3) expected performance during the service life of a bridge; and (4) expected service life assuring both acceptable safety and serviceability levels. The lifetime objectives considered within the optimization procedure include (1) minimizing the expected total cost; (2) maximizing the bridge performance; and (3) maximizing the expected bridge service life, among others.

18.2.2 Bridge Deterioration Mechanisms under Uncertainty

Life-cycle analysis of a bridge structure depends on the assessment and prediction of structural performance considering various deterioration mechanisms under uncertainty (Okasha and Frangopol 2010b, 2011; Akiyama et al. 2011; Frangopol 2011). For this reason, understanding the deterioration mechanisms and accurate performance prediction of a deteriorating bridge are essential for its life-cycle analysis. The deterioration of bridges may be caused by combined effects of progressive structural aging and/or aggressive environmental and mechanical stressors. The most common causes of resistance reduction of concrete and steel structures are corrosion and fatigue (Frangopol et al. 2012).

Corrosion of reinforcement has been considered as one of the critical processes to induce deterioration of reinforced concrete (RC) bridges (Chaker 1992; NCHRP 2005). The deterioration process due to corrosion generally consists of corrosion initiation and propagation (Al-Tayyib and Khan 1988; Dhir et al. 1989; Stewart and Rosowsky 1998; Tuutti 1982). Corrosion initiation can be defined as the time for the chloride concentration at the reinforcement surface to exceed a predefined limit (Arora et al.

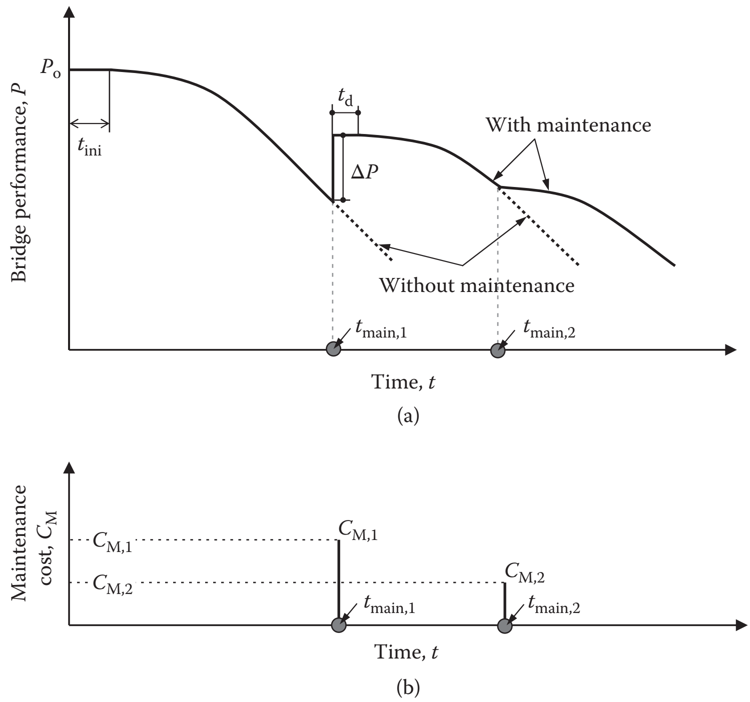


FIGURE 18.1 (a) Time-dependent bridge performance with and without maintenance and (b) associated maintenance cost.

1997; Zhang and Lounis 2006). To predict chloride concentration over time, Fick's second law can be used. The corrosion propagation of the reinforcement in an RC bridge deck, as shown in Figure 18.2a, can be represented by the general and localized corrosion models (Marsh and Frangopol 2008; Val and Melchers 1997). The general corrosion model is associated with uniform reduction of the entire cross-sectional area of reinforcement as shown in Figure 18.2b. According to Gonzalez et al. (1995) and Stewart (2004), corrosion can be highly localized as shown in Figure 18.2c, and the probability of failure associated with localized corrosion is larger than that associated with general corrosion. Detailed information regarding the remaining cross-sectional area at a specific time can be found in Val and Melchers (1997).

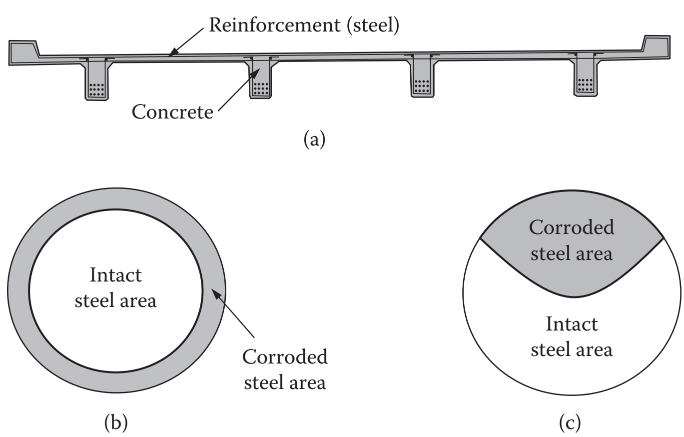


FIGURE 18.2 (a) Cross section of an RC bridge deck, (b) uniform corrosion, and (c) localized corrosion.

Fatigue cracks in steel structures can be initiated and propagated under repetitive loadings. A crack may have occurred during fabrication or initiated by fatigue and/or corrosion (Fisher 1984). Paris' equation has been generally used to predict the crack size over time. The ratio of the crack size increment to the stress cycle increment is described as (Paris and Erdogan 1963)

$$\frac{\mathrm{d}a}{\mathrm{d}N_{\text{cycle}}} = C(\Delta K)^m \quad \text{for } \Delta K > \Delta K_{\text{thres}} \tag{18.1}$$

where a is the crack size, N_{cycle} the cumulative number of cycles, ΔK the stress intensity factor, and ΔK_{thres} the threshold of stress intensity factor. C and m are material parameters. The stress intensity factor ΔK is (Irwin 1958)

$$\Delta K = S_{\text{sr}} Y(a)\sqrt{\pi a} \tag{18.2}$$

where S_{sr} is the stress range and $Y(a)$ denotes the geometry function.

18.2.3 Structural Reliability

The accurate performance prediction of a deteriorating bridge can lead to efficient allocation of the limited maintenance funds for extension of its service life. As mentioned previously, in general, the deterioration mechanisms of a bridge are highly dependent on various environmental and mechanical stressors under uncertainty. Therefore, the structural performance has to be assessed and predicted by using probabilistic concepts and structural reliability theories.

The reliability of a structure is expressed by means of a state function (Ang and Tang 1984) as follows:

$$g(\mathbf{X}) = g(X_1, X_2, \ldots, X_n) \tag{18.3}$$

where $\mathbf{X} = (X_1, X_2, \ldots, X_n)$ is a vector of basic random variables. $g(\mathbf{X})$ determines the state of the structure as follows $[g(\mathbf{X}) > 0]$ = "safe state," $[g(\mathbf{X}) < 0]$ = "failure state," and $[g(\mathbf{X}) = 0]$ = "limit state." The reliability p_{S} and the probability of failure p_{F} are

$$p_{\text{S}} = \int_{g(\mathbf{X})>0} f_{\mathbf{X}}(\mathbf{x})\mathrm{d}\mathbf{x} \tag{18.4a}$$

$$p_{\text{F}} = 1 - p_{\text{S}} = \int_{g(\mathbf{X})<0} f_{\mathbf{X}}(\mathbf{x})\mathrm{d}\mathbf{x} \tag{18.4b}$$

where $f_{\mathbf{X}}(\mathbf{x})$ is the joint probability density function (PDF) of the basic random variables associated with the vector $\mathbf{X}$. These three states are illustrated in Figure 18.3, where the state function consists of the uncorrelated normal variables X_1 and X_2. In the standard normal space, the minimum distance from the origin to the limit state is the reliability index β as shown in Figure 18.3. The standard normal variable X_i' is defined as $(X - \mu_{X_i})/\sigma_{X_i}$, where μ_{X_i} and σ_{X_i} are the mean and standard deviation of X_i. The reliability index β is defined as

$$\beta = \Phi^{-1}(p_{\text{S}}) = \Phi^{-1}(1 - p_{\text{F}}) \tag{18.5}$$

where $\Phi^{-1}(\cdot)$ is the inverse of the standard normal cumulative distribution function.

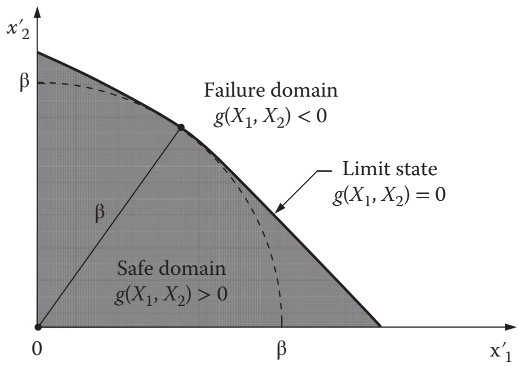


FIGURE 18.3 State function $g(\mathbf{X})$ for safe state, limit state, and failure state in the standard normal space.

18.2.4 System Reliability

The reliability of an existing bridge can be correctly assessed by considering all components (e.g., deck, girder, and pier) and all possible failure modes due to flexure, shear, buckling, and fatigue (Estes and Frangopol 1999). In system reliability analysis, the appropriate modeling can be based on a series system (Figure 18.4a), a parallel system (Figure 18.4b), or a series–parallel system (Figure 18.4c–e). The probabilities of failure p_F of these three systems can be expressed as

$$p_F = p\left(\bigcup_{i=1}^{N}\{g_i(X) \le 0\}\right) \quad \text{for a series system} \tag{18.6a}$$

$$p_F = p\left(\bigcap_{i=1}^{N}\{g_i(X) \le 0\}\right) \quad \text{for a parallel system} \tag{18.6b}$$

$$p_F = p\left(\bigcup_{k=1}^{M}\bigcap_{i=1}^{K}\{g_{i,k}(X) \le 0\}\right) \quad \text{for a series} - \text{parallel system} \tag{18.6c}$$

where N is the number of components in a series or parallel system and M the number of parallel systems in a series–parallel system. The kth parallel system has K components (see Equation 18.6c). The reliability of each component can be estimated by using first-order methods, second-order methods, and Monte Carlo simulation. Figure 18.5 shows failure and safe domains of the three individual components (see Figure 18.5a), the series system (see Figure 18.5b), the parallel system (see Figure 18.5c), and the series–parallel systems (see Figure 18.5d–f). The limit states of the three components and the systems that they comprise are illustrated in Figure 18.6a–c. When X_1 and X_2 are uncorrelated normal variables, the reliability indices β (i.e., the minimum distance from the origin to the limit state in the space of the standard normal variables X'_1 and X'_2) of the components and system models are also presented in these figures. As shown in Figure 18.6d, the reliability index of component 2 is the smallest among the three components (i.e., $\beta_2 < \beta_1 < \beta_3$), and the parallel system modeling provides the largest reliability index among all components and systems. The reliability index of the series system is equal to the reliability index of component 2 and of series–parallel system II (i.e., $\beta_s = \beta_2 = \beta'_{sp}$). Furthermore, series–parallel systems I and III have the same β as that of component 1 (i.e., $\beta_{sp} = \beta_1$). Several computer programs such

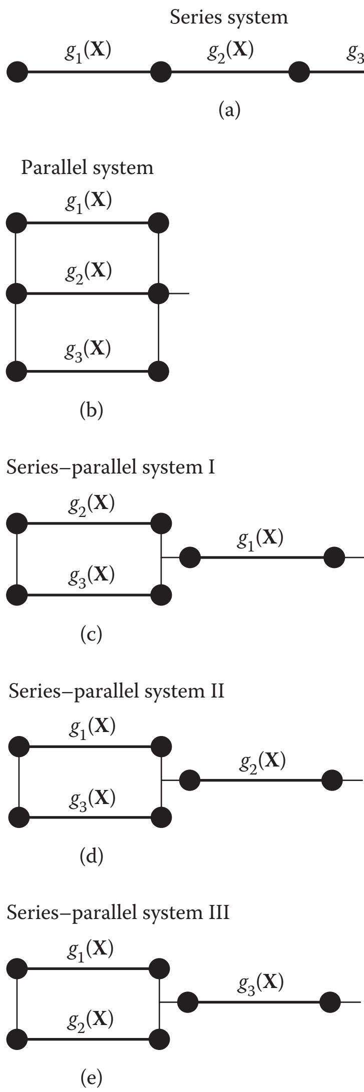


FIGURE 18.4 Modeling of a system with three components: (a) series system; (b) parallel system; (c) series–parallel system I; (d) series–parallel system II; and (e) series–parallel system III.

as CalREL (Liu et al. 1989) and RELSYS (Estes and Frangopol 1998) are available to compute the system reliability considering various types of PDFs and correlations among random variables.

18.2.5 Time-Dependent Reliability

Time-dependent reliability analysis is a rational approach for life-cycle maintenance management of deteriorating bridges under uncertainty. The reliability $p_S(t)$ at time t can be formulated using Equation 18.4a as

$$p_S(t) = \int_{g[\mathbf{X}(t)]>0} f_{\mathbf{X}(t)}[\mathbf{x}(t)]\,d\mathbf{x}(t) \tag{18.7}$$

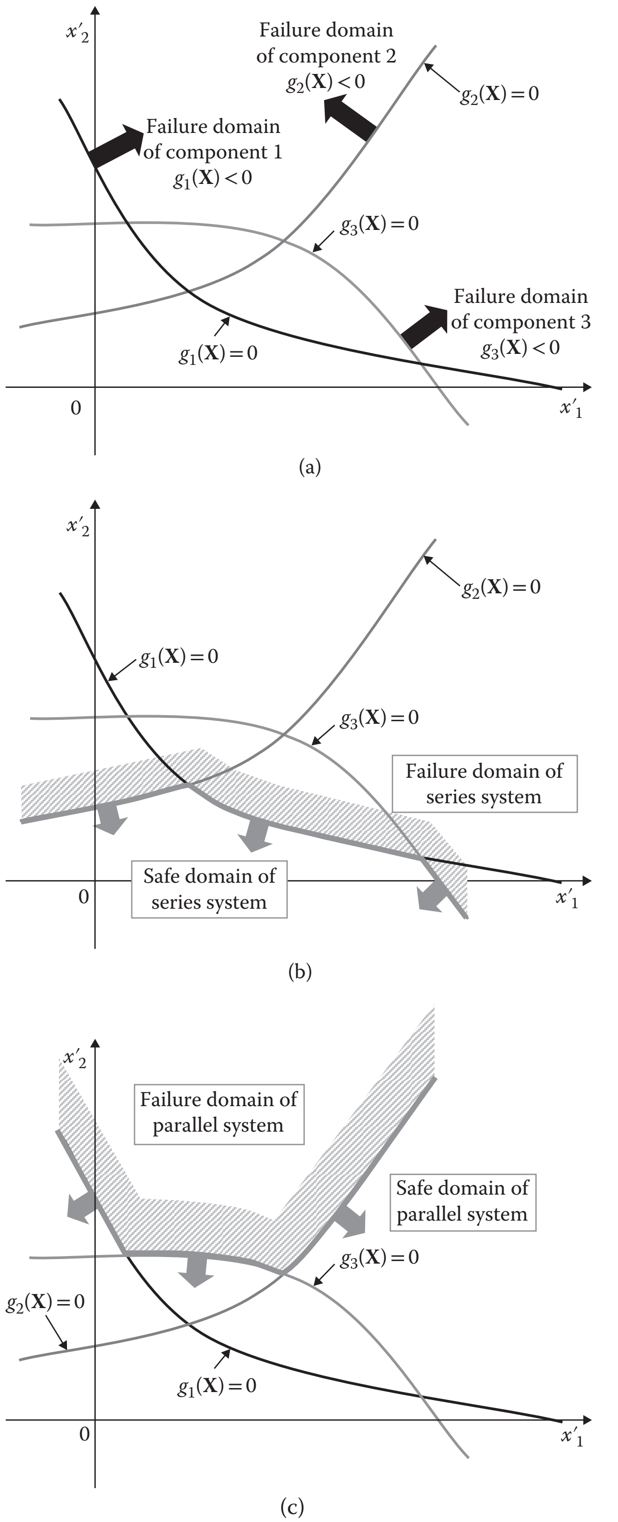


FIGURE 18.5 Failure and safe domains in the standard normal space: (a) three components; (b) series system; and (c) parallel system.

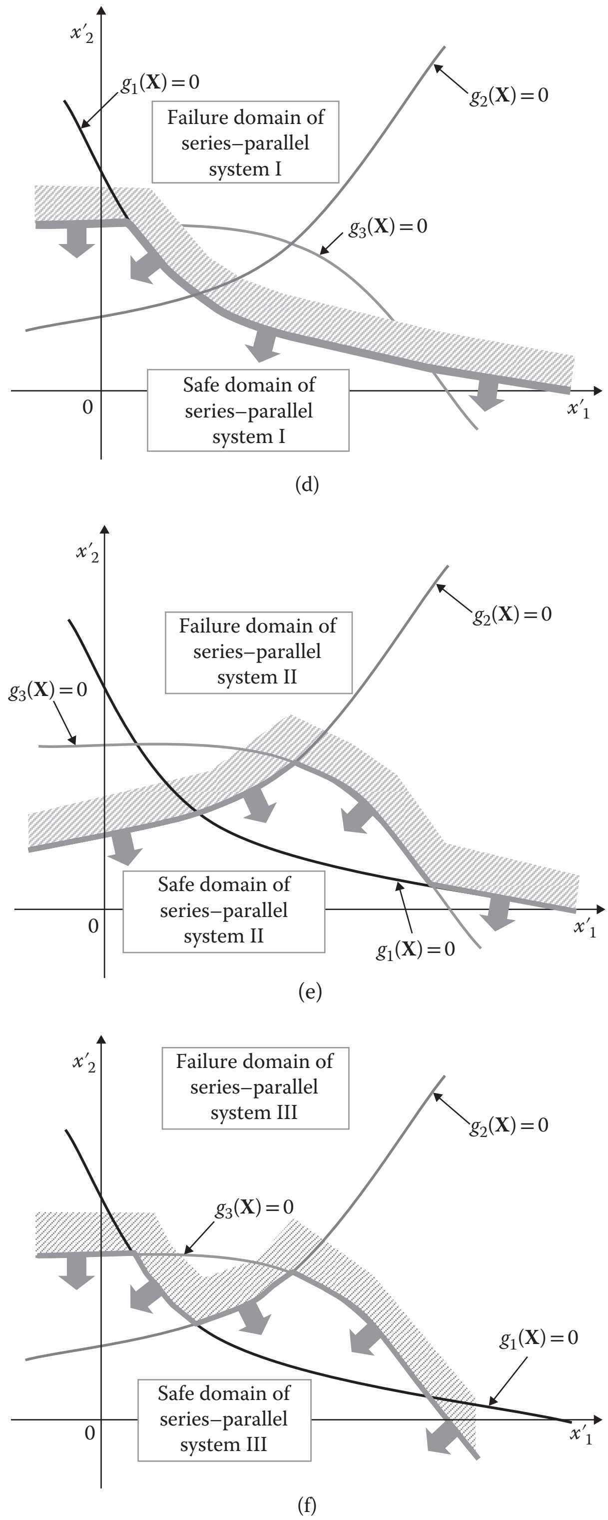


FIGURE 18.5 (*Continued*) Failure and safe domains in the standard normal space: (d) series–parallel system I; (e) series–parallel system II and (f) series–parallel system III

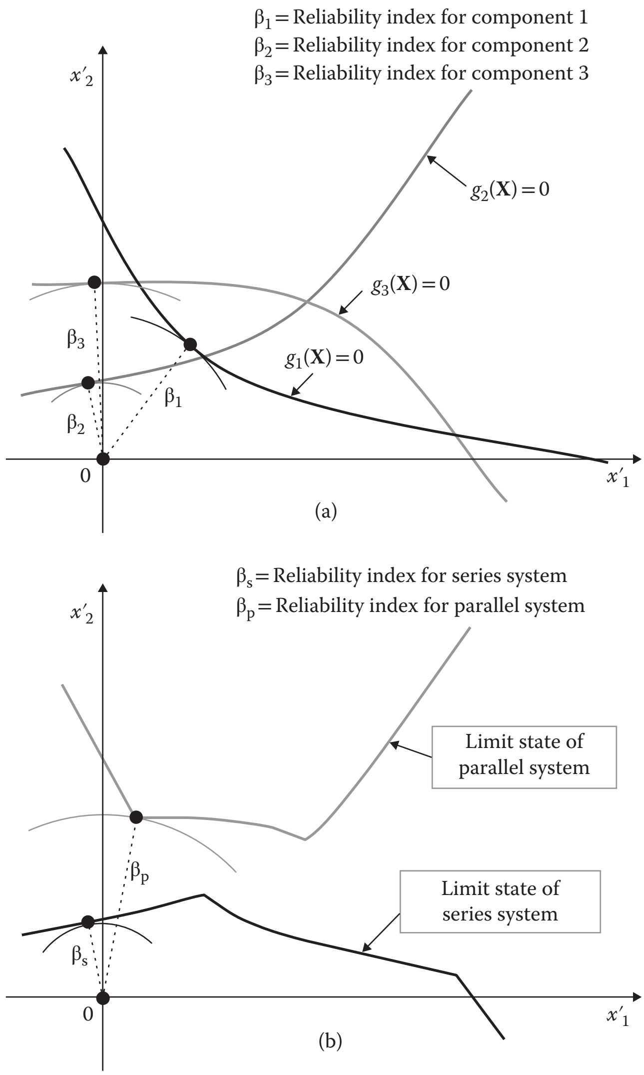
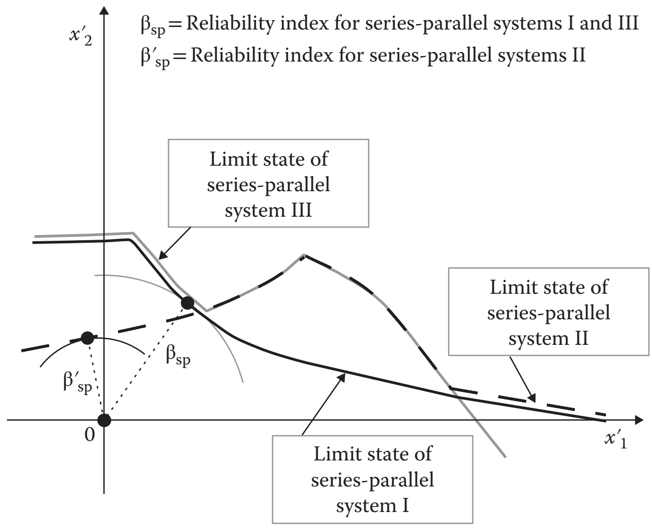
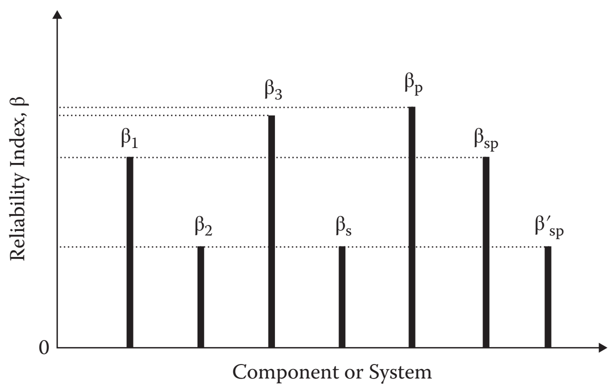


FIGURE 18.6 Reliability indices in the standard normal space: (a) three components and (b) series system and parallel system.

As an example, consider the time-dependent state function $g[X(t)] = X_1(t) - X_2(t)$, where $X_1(t) = Y_1 - t^{2.5}$ and $X_2(t) = Y_2 + t^{2.5}$. The variable Y_1 is normally distributed, and the mean and standard deviation values of Y_1 are 1500 and 150, respectively (denoted as $N(1500; 150)$). Y_1 and Y_2 are uncorrelated. The variable Y_2 is assumed to be normally distributed with $N(800; 160)$. Figure 18.7a shows the mean values of X_1 and X_2 over time, and the PDFs of X_1 and X_2 at time $t = 0$, 4, and 8 years. The associated reliability (i.e., probability of survival) p_S and reliability index β are shown in Figure 18.7b. At time $t = 4$ years, the reliability p_S can be computed using Equation 18.4a. The PDF of $X_1 - X_2$ is also normal. The mean and standard deviation values of $X_1 - X_2$ at $t = 4$ years are 636 (i.e., 1468 – 832) and 219.32 (i.e., $\sqrt{150^2 + 160^2}$), respectively (see Figure 18.7a). The reliability p_S is $\Phi(636/219.32) = 0.9981$, and the associated reliability index β is $636/219.32 = 2.90$ (see Figure 18.7b). The variables X_1 and X_2 in this example could represent the time-dependent resistance and load effect, respectively, associated with a failure mode of a deteriorating bridge.



(c)



(d)

FIGURE 18.6 (*Continued*) Reliability indices in the standard normal space: (c) series–parallel systems I, II, and III and (d) reliability indices associated with (a), (b), and (c).

18.3 Inspection, Monitoring, and Maintenance in Life-Cycle Analysis

Deteriorating bridges are inspected and repaired at uniform or nonuniform time intervals. The purpose of inspection and structural health monitoring (SHM) includes detecting and identifying possible damages, assessing and predicting bridge performance, and providing information for more reliable maintenance planning (Enright and Frangopol 1999; Frangopol et al. 2008). Appropriate inspection and SHM methods should be selected by considering the expected damage types, probability of damage detection, and cost. The maintenance options usually affect the life-cycle performance and cost (NCHRP 2005, 2006). For a cost-effective maintenance strategy, the maintenance types should be selected considering the effect of the maintenance on the structural performance and life-cycle cost of a deteriorating bridge.

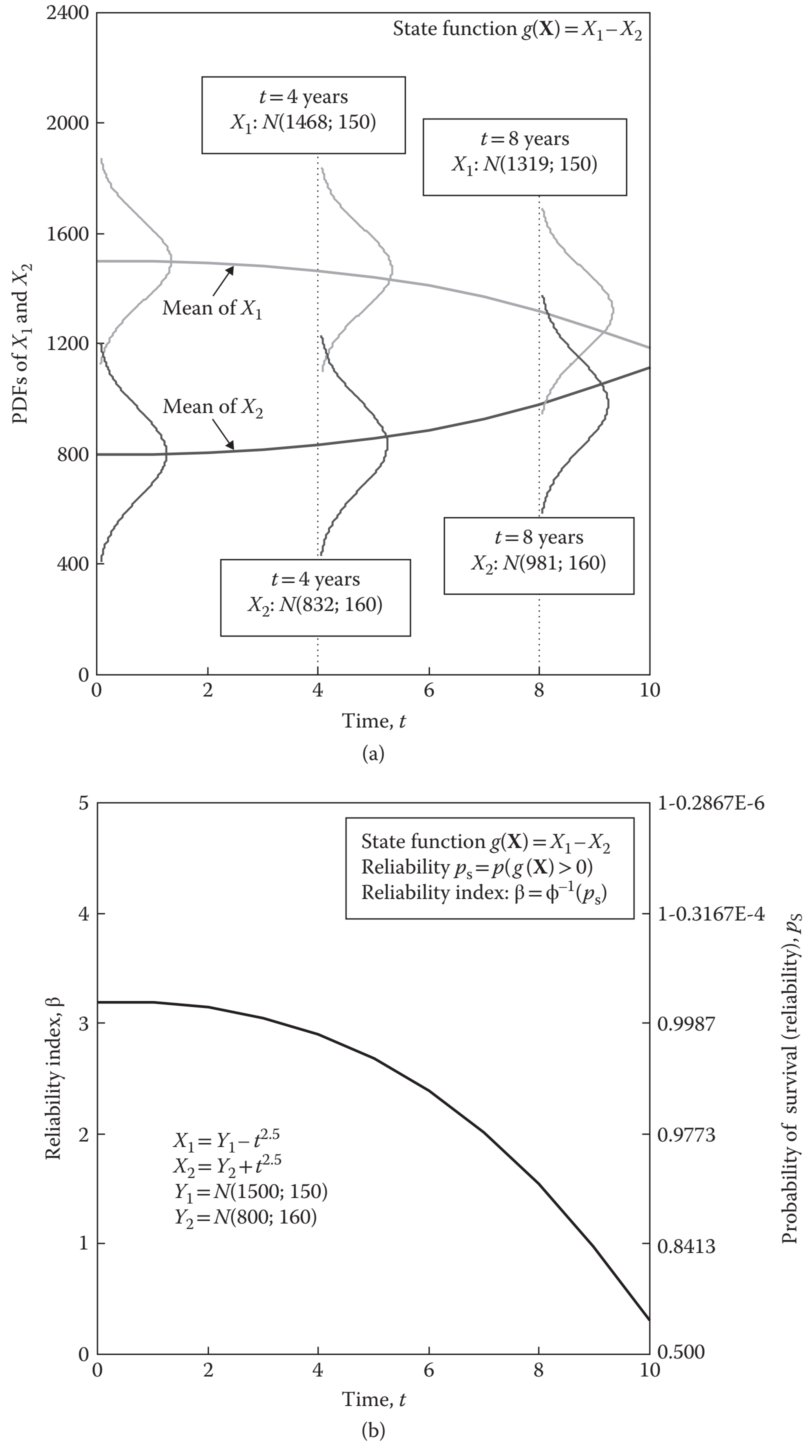


FIGURE 18.7 (a) PDFs of X_1 and X_2 over time and (b) time-dependent probability of survival p_S and reliability index β associated with the state function $g(X) = X_1 - X_2$.

18.3.1 Effects of Inspection and Monitoring on Performance and Service Life Prediction

Inspection methods for deteriorating RC structures under corrosion include (1) visual survey, (2) delamination survey, (3) cover depth measurements, (4) chloride ion content analysis, (5) electrical continuity testing, (6) corrosion potential survey, (7) corrosion rate measurement, and (8) petrographic analysis. The embedded corrosion sensors used to detect corrosion damage can be separated into two categories: physicochemical and electrochemical types of sensors (Montemor et al. 2003).

To detect fatigue crack damage in steel structures, the following inspection methods are commonly used: (1) visual inspection, (2) dye penetrant testing, (3) magnetic particle testing, (4) ultrasonic testing, (5) acoustic emission testing, and (6) X-radiographic testing. Surface-mountable eddy-current, surface acoustic wave, and electrochemical fatigue sensors can also be used for fatigue crack detection (Papazian et al. 2007).

Efficient use of information from inspection and monitoring can lead to cost-effective life-cycle maintenance planning. One goal of bridge life-cycle assessment is to improve the accuracy and reduce the epistemic uncertainty associated with prediction of bridge performance. Figure 18.8 shows the effect of updating bridge performance on service life prediction, using information from inspection and monitoring. If the updated results indicate that the bridge performance and deterioration rate are less and higher than predicted initially, respectively, then the updated predicted service life $t_{life,a}$ is shorter than the initially predicted service life $t_{life,o}$. Case A (see Figure 18.8) exhibits these properties and shows that inaccurate prediction of bridge performance can endanger the survival of the structure. Case B indicates that the initially predicted service life $t_{life,o}$ can be shorter than the updated predicted service life $t_{life,b}$; therefore, unnecessary maintenance actions can be avoided through performance updating based on the information obtained from inspection and monitoring (Frangopol 2011).

18.3.2 Effects of Maintenance on Life-Cycle Performance

Maintenance actions are categorized as preventive or essential (Das 1999). Preventive maintenance is applied at predefined times, to delay the deterioration process and to keep the bridge above the required level of structural performance (Estes and Frangopol 2005). If maintenance is done after the scheduled

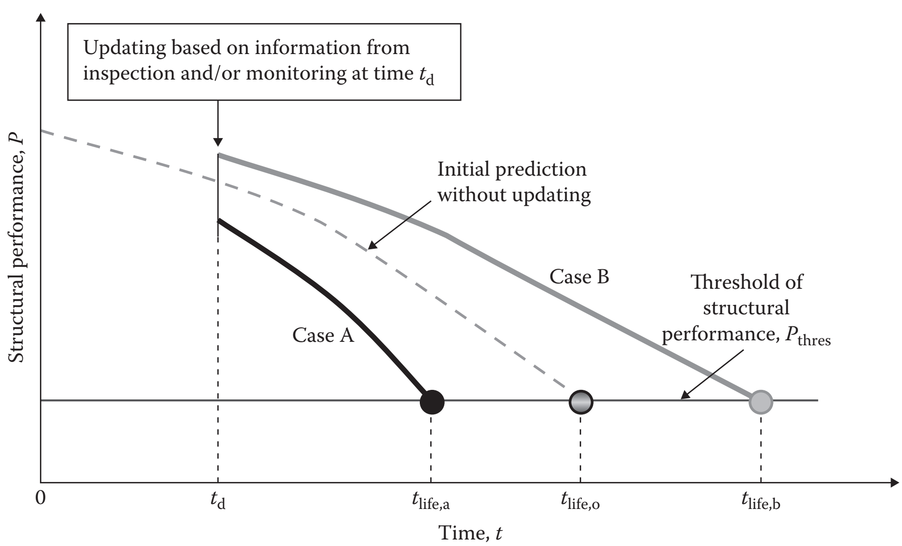


FIGURE 18.8 Effect of updating structural performance using information from inspection and/or monitoring on the service life prediction of a deteriorating structure.

time, additional costs will be required to maintain bridge safety levels. Preventive maintenance for a deteriorating bridge includes replacing small parts, patching concrete, repairing cracks, changing lubricants, and cleaning or painting exposed parts, among others. Essential maintenance is a performance-based intervention. Figure 18.9a shows the effects of preventive and essential maintenance actions on the structural performance and the service life of a bridge. As shown in this figure, essential maintenance is applied when the bridge performance P reaches a predefined threshold P_{thres}. Essential maintenance actions can lead to higher levels of bridge performance than preventive maintenance actions, but they usually cost more. Strengthening and replacement of bridge components are examples of essential maintenance actions.

The initial bridge performance and the improvement of the performance after preventive or essential maintenance are not deterministic as shown in Figure 18.9b. Therefore, the service life with or without the maintenance should be predicted using probabilistic concepts and methods. Using the lifetime function and multiobjective optimization, a rational approach to establish the optimum essential and/or preventive maintenance strategies for a bridge structure was proposed by Okasha and Frangopol (2010a). Bocchini and Frangopol (2011a) and Frangopol and Bocchini (2012) developed a probabilistic computational framework for optimal highway bridge maintenance planning at network level.

18.4 Life-Cycle Optimization

Efficient bridge life-cycle management requires optimization (Frangopol 2011; Barone et al. 2013). Through this process, the optimum application times and types of maintenance actions can be obtained. The optimization of bridge structures is based on objectives related to structural performance indicators (e.g., condition, reliability, risk, robustness, and redundancy), total cost, and service life. Considering the interaction among objectives, a multiobjective optimization can be formulated and solved. Among all possible solutions obtained from the multiobjective optimization, the most appropriate solutions can be selected.

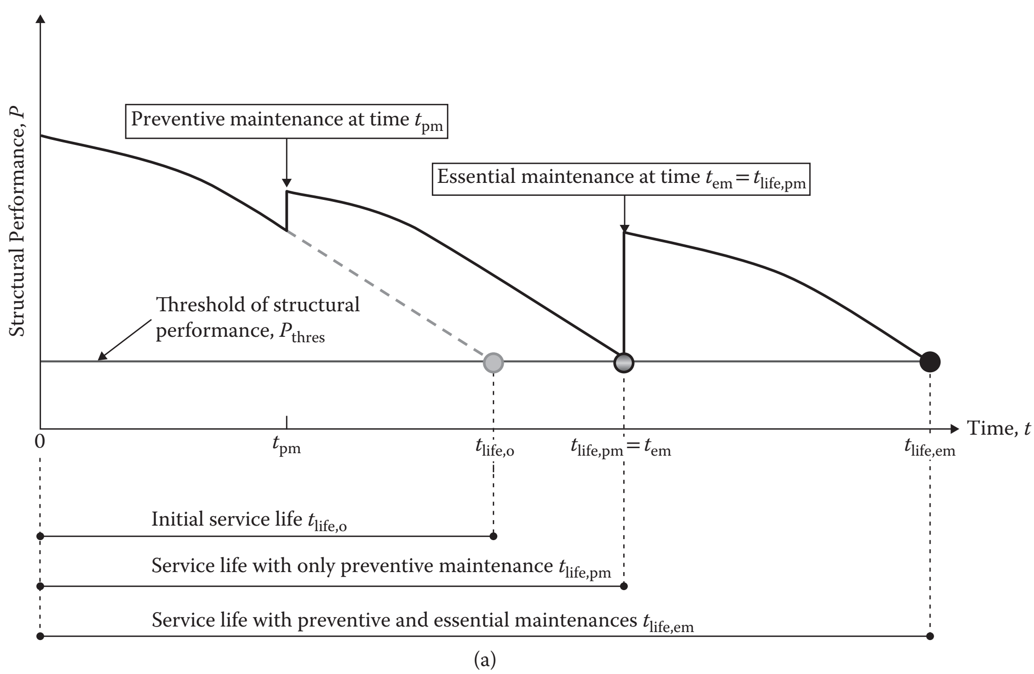
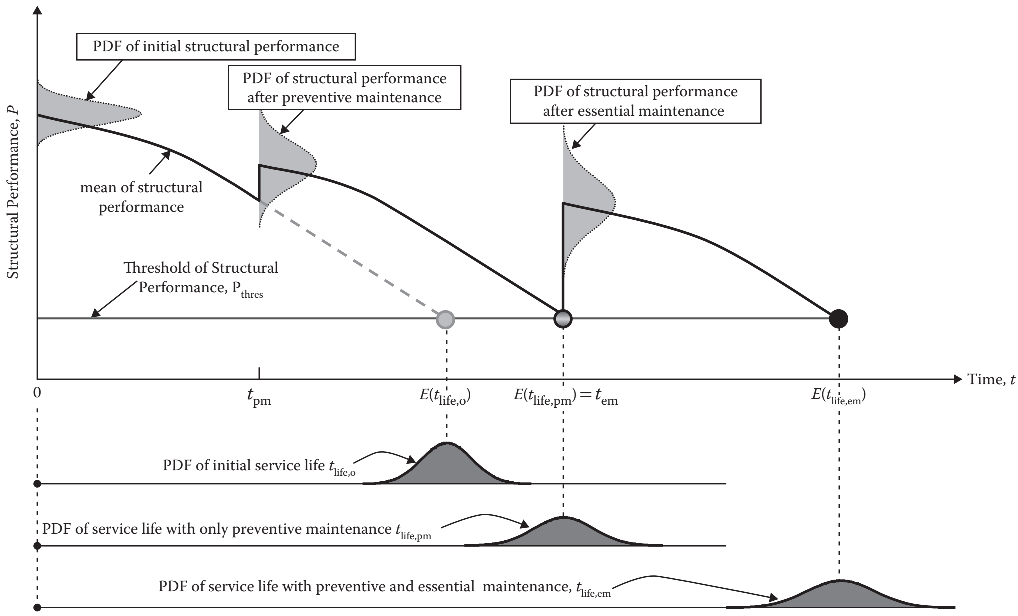


FIGURE 18.9 Predictions of bridge performance and the associated service life with and without maintenance based on (a) deterministic approach.



(b)

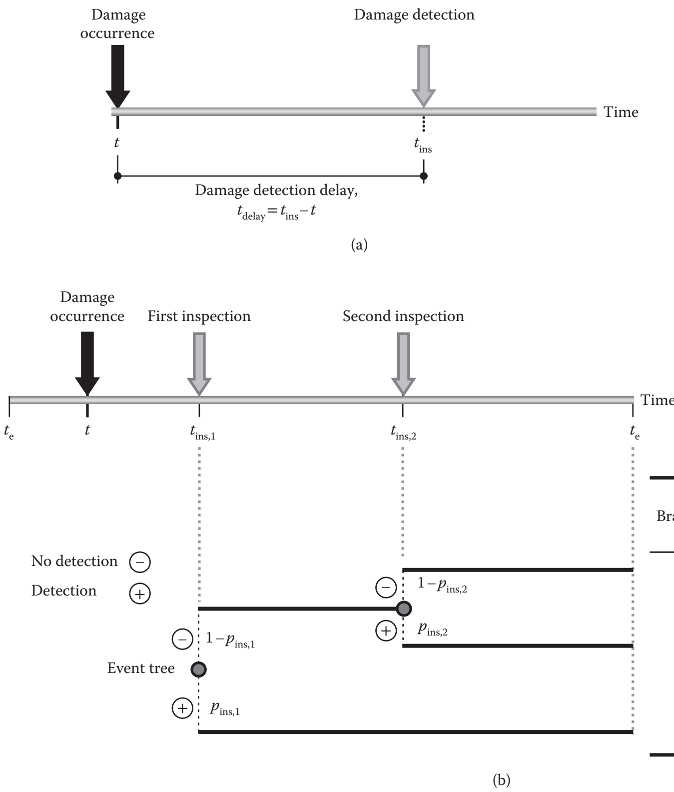
FIGURE 18.9 (*Continued*) Predictions of bridge performance and the associated service life with and without maintenance based on (b) probabilistic approach.

18.4.1 Optimum Inspection Planning under Uncertainty

Maintenance actions generally depend on the results obtained from inspections and/or monitoring (Farhey 2005). If damage is not detected and identified, effective and timely maintenance actions cannot be applied. Therefore, to prevent unexpected performance and to extend the service life of a bridge structure, the time delay from damage occurrence to damage detection has to be minimized and, furthermore, maintenance delay should be reduced (Kim and Frangopol 2011a). In this context, optimum inspection planning can be based on minimizing the expected damage detection delay. Uncertainties associated with damage occurrence/propagation and damage detection have to be included in the formulation of the expected damage detection delay.

As shown in Figure 18.10a, damage detection delay t_{delay} is defined as the time interval between the damage occurrence time t and the time t_{ins} for the damage to be detected (Huang and Chiu 1995). Using the event tree model and considering probability of damage detection and number of inspections, the expected damage detection delay can be formulated. This model is the binary logic model that identifies all the possible outcomes resulting from an initial event and quantifies the associated probabilities.

For example, assuming that inspections are performed at times $t_{ins,1}$ and $t_{ins,2}$ in order to detect damage that occurs when time $t < t_{ins,1}$, and that damage is detected when time $t_e > t_{ins,2}$, the event describing this situation can be developed (see Figure 18.10b). Figure 18.10b indicates that two mutually exclusive events (i.e., detection and no detection) are considered at both inspection times (denoted by gray shaded circles). Consequently, this event tree consists of three branches. Each branch is represented by two parameters: the damage detection delay and its occurrence probability. For instance, as shown in Figure 18.10b, branch 2 corresponds to the event that damage is not detected at the first inspection time $t_{ins,1}$, but is detected at the second inspection time $t_{ins,2}$. The associated damage detection delay and its occurrence probability are $t_{ins,2} - t$ and $(1 - p_{ins,1}) \times p_{ins,2}$, respectively, where $p_{ins,i}$ denotes the probability of damage detection associated with the ith inspection method. Considering all three branches, the



Branch	Damage Detection Delay	Branch Probability
1	$t_e - t$	$(1-p_{ins,1}) \times (1-p_{ins,2})$
2	$t_{ins,2} - t$	$(1-p_{ins,1}) \times p_{ins,2}$
3	$t_{ins,1} - t$	$p_{ins,1}$

FIGURE 18.10 (a) Damage detection delay and (b) event tree with two inspections.

associated damage detection delays, and occurrence probabilities, the expected damage detection delay $E(t_{delay})$ is formulated as

$$E\left(t_{delay}\right) = \left(t_{ins,1} - t\right) \cdot p_{ins,1} + \left(t_{ins,2} - t\right)\left[\left(1 - p_{ins,1}\right) \cdot p_{ins,2}\right] + \left(t_e - t\right)\left[\left(1 - p_{ins,1}\right)\left(1 - p_{ins,2}\right)\right] \quad (18.8)$$

If N_{ins} inspections are used to detect the damage, the number of branches is equal to $N_{ins} + 1$, and the expected damage detection $E(t_{delay})$ is (Kim and Frangopol 2011a)

$$E\left(t_{delay}\right) = \sum_{i=1}^{N_{ins}+1} \left[p_{branch,i} \cdot t_{delay,i}\right] \quad (18.9)$$

where $p_{branch,i}$ and $t_{delay,i}$ are the occurrence probability and damage detection delay associated with branch i, respectively.

Furthermore, considering the uncertainty related to the damage occurrence time t, the expected damage detection delay $E(t_{delay})$ for N_{ins} inspections is expressed as (Kim and Frangopol 2011a)

$$E\left(t_{delay}\right) = \sum_{i=1}^{N_{ins}+1} \left\{ \int_{t_{ins,i-1}}^{t_{ins,i}} \left[E\left(t_{delay}\right)_{case,i} \cdot f_T\left(t\right)\right] dt \right\} \quad (18.10)$$

where $E(t_{\text{delay}})_{\text{case},i}$ is the expected damage detection delay, when the damage occurs between the time associated with the $(i-1)$th inspection and that of the ith inspection (i.e., $t_{\text{insp},i-1} \leq t \leq t_{\text{insp},i}$); $t_{\text{insp},0}$ for $i = 1$ and $t_{\text{insp},N_{\text{ins}}}$ for $i = N_{\text{ins}} + 1$ are t_s and t_e, respectively; and $f_T(t)$ is the PDF of the damage occurrence time.

As an example, consider the deck of an RC bridge under corrosion assuming discrete and continuous damage occurrence times.

(1) *Expected damage detection time for discrete corrosion damage occurrence time*: The corrosion damage can occur at 4 and 8 years. t_s and t_e are assumed to be 0 and 15 years, respectively. When two inspections with the probability of detection of 0.9 are applied at 5 and 10 years, the expected damage detection delay can be obtained using Equation 18.9. Two cases (i.e., case 1 and case 2 as shown in Figure 18.11) are considered for the time it takes corrosion damage to occur. For the first

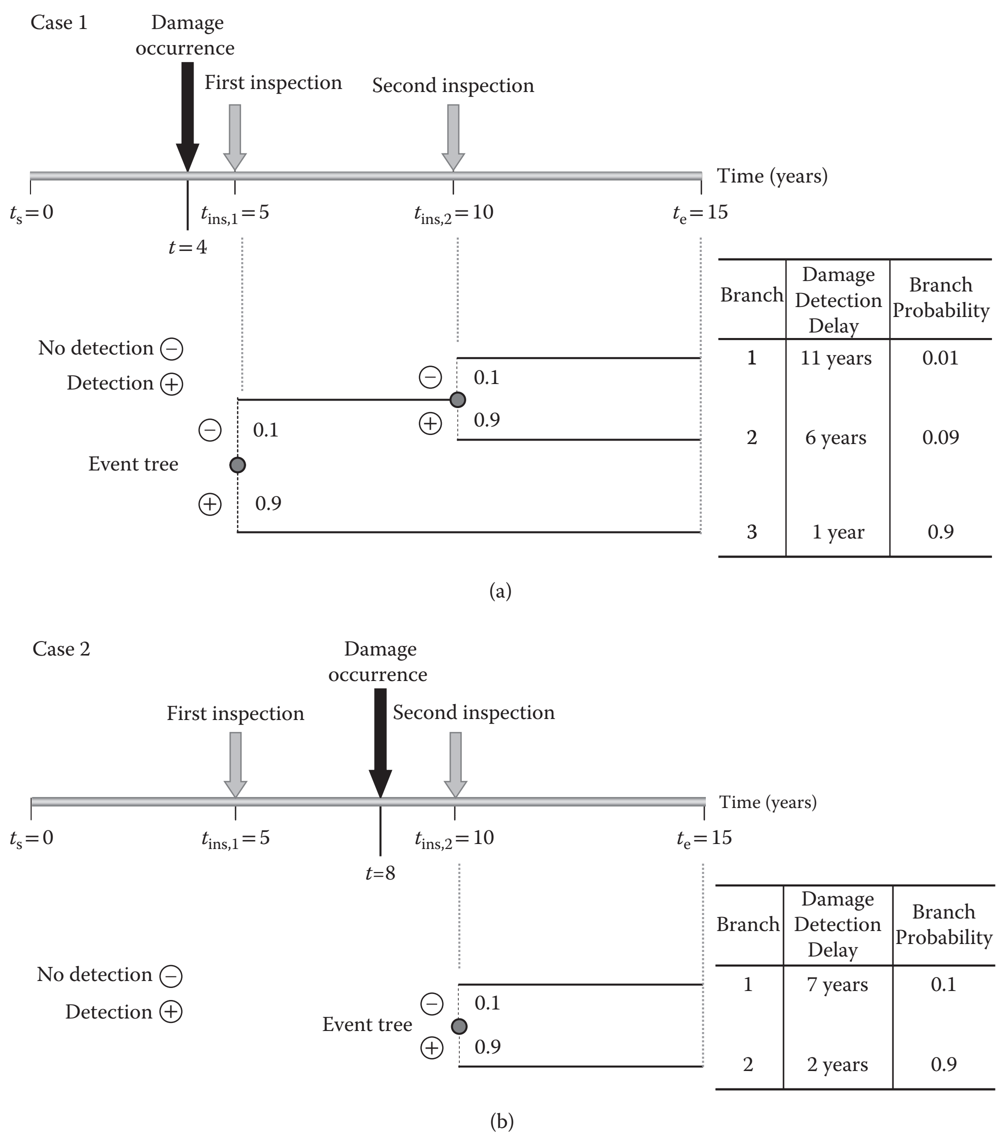


FIGURE 18.11 Event tree: (a) case 1: $t_s \leq t < t_{\text{ins},1}$ and (b) case 2: $t_{\text{ins},1} \leq t < t_{\text{ins},2}$.

case (i.e., the corrosion damage occurs at time $t = 4$ years), the expected damage detection delay $E(t_{delay})$ is $(11 \times 0.01) + (6 \times 0.09) + (1 \times 0.9) = 1.55$ years (see Equation 18.9 and Figure 18.11a). Similarly, $E(t_{delay})$ for case 2 can be obtained as 2.5 years (see Figure 18.11b).

(2) *Expected damage detection time for continuous corrosion damage occurrence time*: When the corrosion damage occurs between $t_s = 0$ year and $t_e = 15$ years, and two inspections associated with the probability of detection of 0.9 are applied at 5 and 10 years, the expected damage detection delay can be obtained using Equation 18.10. For uniformly distributed damage occurrence time (see Figure 18.12a), the expected damage detection delay $E(t_{delay})$ is 2.85 years. When the PDF of corrosion damage occurrence time is triangular as shown in Figure 18.12b (i.e., $f_T(t) = 2 \times t/225$ in Equation 18.10), $E(t_{delay})$ is 2.45 years.

To find the optimum inspection times to minimize the expected damage detection delay, optimization techniques are required. The formulation of the optimization problem is

$$\text{Find} \qquad t_{ins,1} \text{ and } t_{ins,2} \tag{18.11a}$$

$$\text{to minimize} \qquad E\left(t_{delay}\right) \tag{18.11b}$$

$$\text{subject to} \qquad t_s \le t_{ins,1} < t_{ins,2} \le t_e \tag{18.11c}$$

The objective of the optimization problem is to minimize the expected damage detection delay $E(t_{delay})$ as indicated in Equation 18.11b. The design variables are the inspection times $t_{ins,1}$ and $t_{ins,2}$ (see Equation 18.11a). To solve this problem, the optimization toolbox provided in MATLAB® version R2009a (MathWorks Inc. 2009) was used. When the PDF of damage occurrence time $f_T(t)$ has the uniform

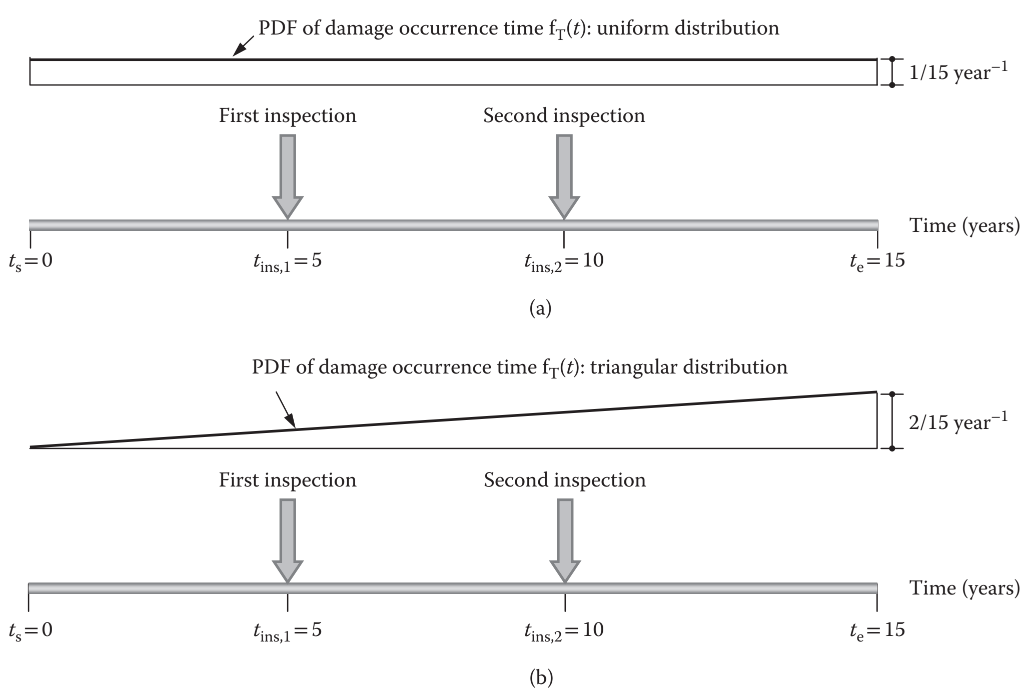


FIGURE 18.12 Inspection plan with two inspections at $t_{ins,1} = 5$ years and $t_{ins,2} = 10$ years: PDF of damage occurrence time $f_T(t)$ is (a) uniform and (b) triangular.

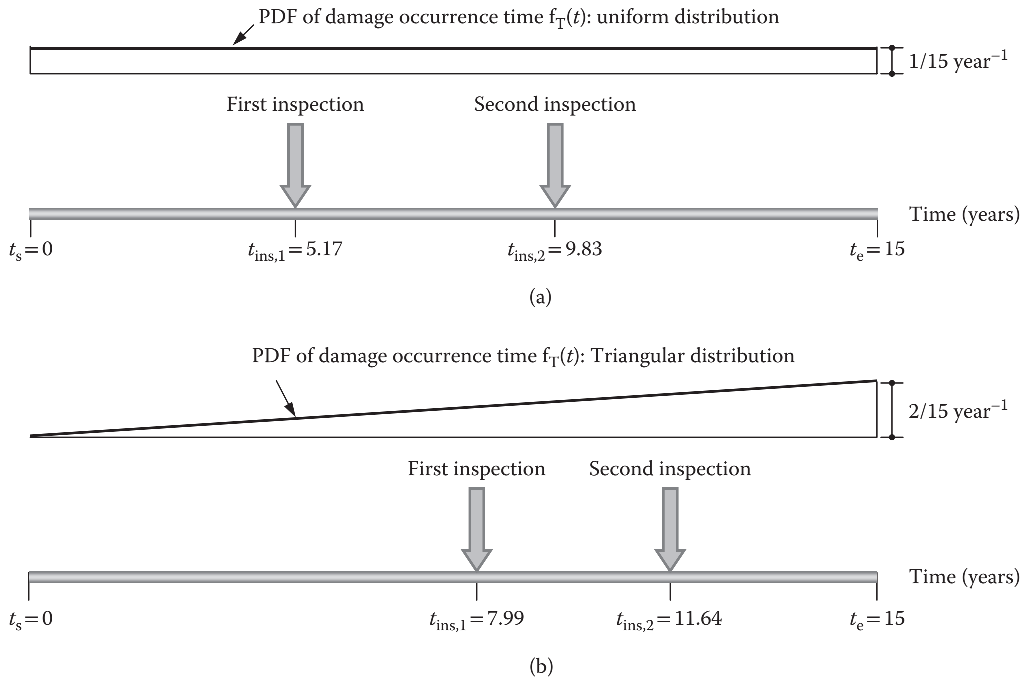


FIGURE 18.13 Optimum inspection plan with two inspections: PDF of damage occurrence time $f_T(t)$ is (a) uniform and (b) triangular.

distribution with $t_s = 0$ year and $t_e = 15$ years, the inspections should be applied at 5.17 and 9.83 years to minimize $E(t_{delay})$ (see Figure 18.13a), and the associated $E(t_{delay})$ is 2.84 years. Furthermore, the optimum inspection times for triangular distributed damage occurrence time are 7.99 and 11.64 years (see Figure 18.13b), and the associated $E(t_{delay})$ is 2.16 years.

In general, the probability of damage detection is related to damage propagation and the quality of inspection. Studies considering this relation can be found in Kim and Frangopol (2011a,b), where the effects of number and quality of inspections on the expected damage detection delay were investigated.

18.4.2 Optimum Maintenance Planning to Minimize Total Expected Cost

Safety of a deteriorating bridge should be cost-effectively maintained during its lifetime. Minimization of life-cycle cost has been the most widely used criterion for life-cycle optimization (Frangopol 1995; Frangopol and Liu 2006). The general formulation of the expected life-cycle cost C_{ET} during a predefined service life of a bridge is (Frangopol et al. 1997a)

$$C_{ET} = C_{INI} + C_M + C_{INS} + C_{FAIL} \tag{18.12}$$

where C_{INI} is the initial cost (i.e., design and construction cost), C_M the cost of maintenance, C_{INS} the cost of inspection, and C_{FAIL} the expected cost of failure. C_{FAIL} is expressed as

$$C_{FAIL} = p_{F,life} \times C_f \tag{18.13}$$

where $p_{F,life}$ is the lifetime probability of failure and C_f the expected monetary loss due to structural failure. C_f should be estimated in a rational way by considering the cost of design and construction of a new bridge, expected cost of human injuries, and user costs, among others (Estes and Frangopol 2005).

As an example, it is assumed that the time-dependent reliability of a bridge is expressed as $p_S(t) = e^{-0.000045t}$, C_{INI} and C_{INS} are 100 and 5, respectively (see Equation 18.12), and the expected monetary loss is $C_f = 10{,}000$ in Equation 18.13. Two maintenance actions are applied at time $t_{main,1}$ and $t_{main,2}$, and the probability of failure p_F is decreased to 10^{-6} after each maintenance. The maintenance cost is estimated considering the relation between the increase of the reliability and associated cost as $C_M = C_{INI} \times p'_F(t_{main})$, where $p'_F(t_{main})$ is the probability of failure before maintenance at time t_{main}. The lifetime probability of failure $p_{F,life}$ is defined as (Frangopol et al. 1997a)

$$p_{F,life} = \max\left[p'_F\left(t_{main,1}\right), p'_F\left(t_{main,2}\right), p_F\left(t_{life}\right)\right] \tag{18.14}$$

where $p'_F(t_{main,i})$ is the probability of failure before ith maintenance application and $p_F(t_{life})$ the probability of failure at the predefined service life t_{life} (i.e., 50 years). The optimum maintenance plan is the solution of the minimization of the expected total cost C_{ET} (see in Equation 18.12) during the predefined service life t_{life}:

$$\text{Find} \quad t_{main,1} \text{ and } t_{main,2} \tag{18.15a}$$

$$\text{to minimize} \quad C_{ET} \tag{18.15b}$$

$$\text{subject to} \quad p_{F,life} \le p_{F,thres} \tag{18.15c}$$

The design variables are the first and second maintenance application times $t_{main,1}$ and $t_{main,2}$ as indicated in Equation 18.15a. The lifetime probability of failure $p_{F,life}$ has to be at most the threshold probability of failure $p_{F,thres}$ ($p_{F,thres} = 0.001$ herein). Figure 18.14 shows the time-dependent probability of failure $p_F(t)$ associated with optimum maintenance application times. The first and second maintenance applications have to be performed at 16.67 and 33.33 years to minimize C_{ET} during 50 years as shown in this figure, and the associated cost C_{ET} is equal to 210.15.

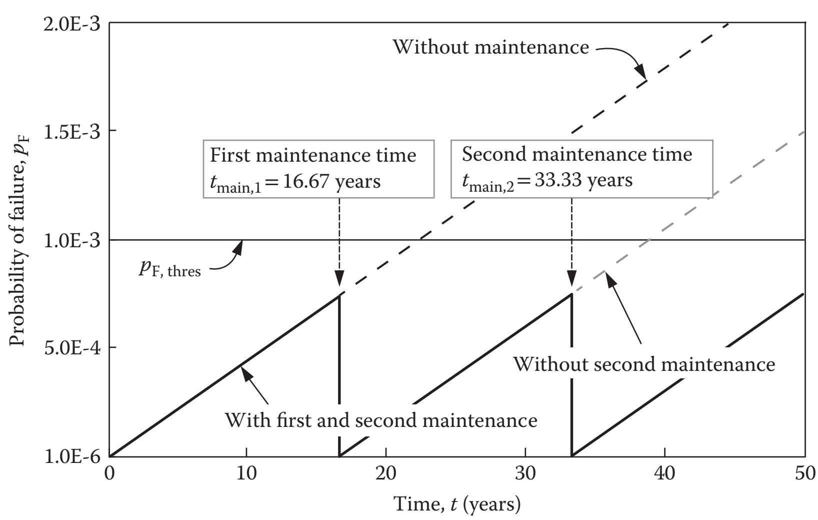


FIGURE 18.14 Optimum maintenance plan to minimize the expected total cost.

Furthermore, Frangopol et al. (1997a) proposed a probabilistic approach including the probability of damage detection, and effects of failure cost, quality, and number of inspections on maintenance planning. More advanced probabilistic approaches to establish the optimum maintenance planning considering various types of maintenance actions have been investigated by Neves et al. (2006b) and Okasha and Frangopol (2010a).

18.4.3 Optimum Maintenance Planning to Maximize Service Life

Service life of deteriorating bridge structures can be extended through appropriate maintenance actions. Considering the improvement of bridge performance and extension of service life after maintenance actions as indicated in Figure 18.9, an optimum maintenance plan can be established (Kim et al. 2011).

A simplified example follows. It is assumed that the reliability p_S of a bridge decreases over time as $p_S(t) = e^{-0.000045t}$. The service life of this bridge is defined as the time when the reliability p_S reaches the threshold $p_{S,thres}$ of 0.999. The service life $t_{life,o}$ without any maintenance is $\ln(p_{S,thres})/(-0.000045) =$ 22.23 years. It is also assumed that two maintenance actions are performed, and that probability of failure p_F decreases to 10^{-6} after each maintenance. The optimization problem for optimum maintenance planning is formulated as

$$\text{Find} \quad t_{main,1} \text{ and } t_{main,2} \tag{18.16a}$$

$$\text{to maximize} \quad t_{life} \tag{18.16b}$$

$$\text{subject to} \quad p_{F,life} \le p_{F,thres} \tag{18.16c}$$

The design variables of this problem are the first and second maintenance application times $t_{main,1}$ and $t_{main,2}$ (see Equation 18.16a), and the objective is to maximize the service life (see Equation 18.16b). The constraint is indicated in Equation 18.16c (i.e., the lifetime probability of failure $p_{F,life}$ has to be less than or equal to the threshold $p_{F,thres}$ of 0.001). The lifetime probability of failure $p_{F,life}$ can be obtained using Equation 18.14. The optimum maintenance plan is illustrated in Figure 18.15. The first and second maintenance should be applied at 22.23 and 44.46 years, respectively, to maximize the service life t_{life}. The associated t_{life} is 66.70 years. From this result, it can be seen that for maximizing the service life, the maintenance should be performed when the reliability p_F reaches the threshold $p_{F,thres}$. The expected total cost C_{ET} during the maximum service life t_{life} of 66.70 years is 212.7 based on the assumptions associated with costs in the earlier example of Section 18.4.2.

As mentioned previously, the maintenance actions can be applied when inspection results identify damage. Furthermore, the maintenance action depends on the decision maker's attitude toward risk. If damage is detected, the decision maker should decide whether immediate repair is required. This attitude toward risk can be considered as the probability of applying maintenance when damage is detected (Estes and Frangopol 2001). Kim et al. (2011) considered the relation between probabilities of damage detection and maintenance, and proposed a probabilistic approach for optimum inspection/maintenance planning to maximize the service life of deteriorating bridges.

18.4.4 Reliability-Based Optimum Life-Cycle Bridge Maintenance Planning: A Case Study

Life-cycle analysis of an existing bridge requires its assessment and prediction under uncertainty, and consideration of the effects of maintenance actions and their associated uncertainties on bridge performance and cost (Frangopol et al. 2001; Furuta et al. 2011). The reliability index profile has been used for

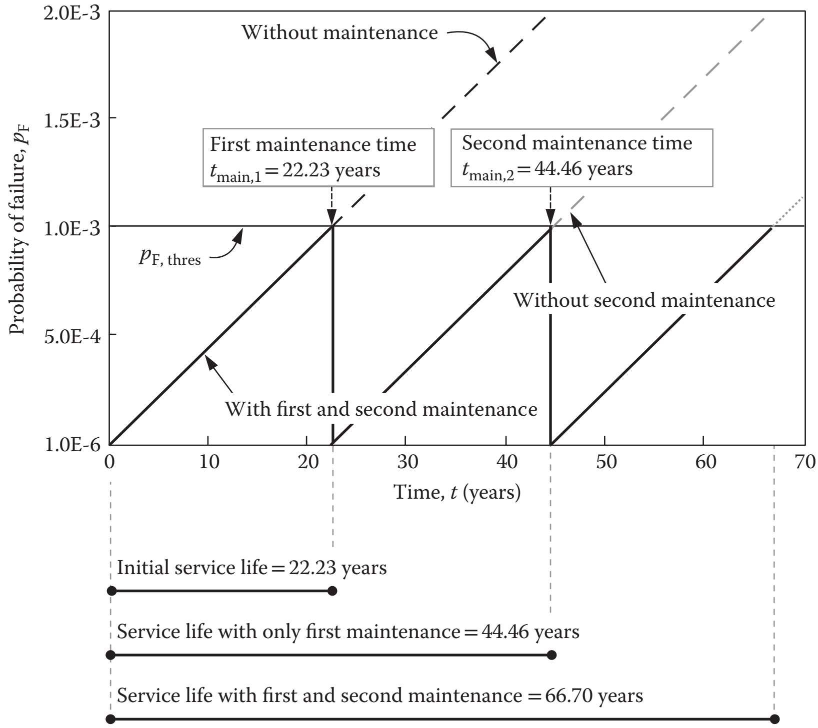


FIGURE 18.15 Optimum maintenance plan to maximize the service life.

optimum maintenance management of deteriorating bridges under uncertainty (Frangopol et al. 2001; Kong and Frangopol 2003, 2004; Petcherdchoo et al. 2008).

A case study is conducted on the existing Colorado Highway Bridge E-17-LE. A detailed description of this bridge is available in Akgül (2002). Under no maintenance action, the reliability index profile of the slab of the bridge is (Frangopol et al. 2001)

$$\beta(t) = \beta_0 \quad \text{for} \quad 0 \le t \le t_{ini} \tag{18.17a}$$

$$\beta(t) = \beta_0 - \alpha(t - t_{ini}) \quad \text{for} \quad t_{ini} \le t \tag{18.17b}$$

where β_0 is the initial reliability index, t_{ini} the time of damage initiation (years), and α the deterioration rate (year^{-1}). Considering the uncertainties associated with the reliability index profile, probabilistic life-cycle analysis of the bridge slab can be performed. The random variables β_0, t_{ini}, and α are assumed to have a triangular PDF with minimum, mode, and maximum [2.17; 2.88; 3.59], [10; 15; 20], and [0.058; 0.077; 0.096], respectively (Frangopol et al. 2001, Petcherdchoo et al. 2008). It is assumed that the maintenance action (i.e., slab replacement) leads to the initial reliability index of the slab as shown in Figure 18.16a and only one maintenance action is applied. Optimum maintenance planning consists of maximizing the mean service life of the slab. The service life t_{life} of the bridge slab is defined as the time when the reliability index β reaches its threshold (i.e., $\beta_{thres} = 2.0$). The formulation of the optimization problem is as follows:

$$\text{Find} \qquad t_{main} \tag{18.18a}$$

$$\text{to maximize} \qquad E(t_{life}) \tag{18.18b}$$

The design variable of this problem is the maintenance application time t_{main}. Monte Carlo simulation with a sample size of 100,000 is used to predict the reliability index based on Equation 18.17. The objective is to maximize the mean service life $E(t_{life})$. The optimum value of t_{main} is 20.81 years and the associated $E(t_{life})$ is 45.30 years. Figure 18.16b illustrates the change in the predicted reliability index distribution over time when the maintenance action is applied at the optimum time of 20.81 years. Further studies of reliability-based optimum maintenance planning considering a series of maintenance interventions and cost can be found in Frangopol et al. (2001), Kong and Frangopol (2003, 2004), and Petcherdchoo et al. (2008).

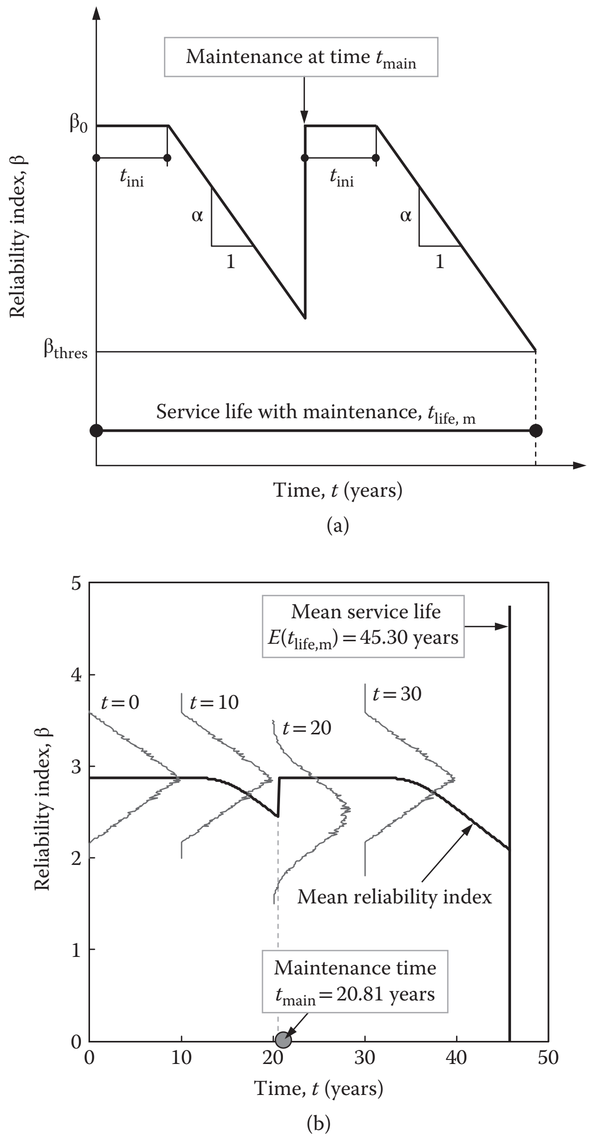


FIGURE 18.16 (a) Reliability index profile with maintenance and (b) probabilistic reliability index profile associated with optimum maintenance to maximize the service.

18.4.5 Multiobjective Life-Cycle Optimization

Optimum life-cycle bridge management depends on the objective of the optimization formulation. The objectives can be related to each other or independent. Consideration of the relation among the objectives can lead to more rational and flexible decisions (Frangopol 2011). For example, increase in the service life of a bridge leads to additional financial expenditures. It means that maximizing the service life conflicts with minimizing the expected total cost. The resulting multiobjective optimization provides a set of well-balanced solutions that allow decision makers to have much flexibility in selecting the best compromise (Liu and Frangopol 2005b).

As shown in Figure 18.17, the boundary of the feasible criterion space (i.e., Pareto optimal set) depends on the objectives used in the optimization formulation. When the bi-objective optimization has two objective functions f_1 and f_2 to be minimized, the Pareto optimal set in the criterion space is shown in Figure 18.17a. The Pareto optimal set associated with maximization of f_1 and minimization of f_2 is shown in Figure 18.17b. Figure 18.17c shows the Pareto optimal set of the optimization with objectives of maximizing both f_1 and f_2. To solve the multiobjective optimization problem, the following approaches can be used: (1) weighted sum, (2) weighted min–max, (3) weighted global criterion, (4) ε-constraint, and (5) genetic algorithm (Arora 2004).

Several life-cycle management approaches based on multiobjective optimization have been proposed. Table 18.1 summarizes the recent studies using multiobjective optimization for life-cycle optimization (including bridge maintenance planning at bridge component level and network level, and inspection and/or monitoring planning). As indicated in Table 18.1, bridge condition index, safety index, and life-cycle maintenance cost are used to formulate the multiobjective optimization problem in Liu and Frangopol (2005a,b), Neves et al. (2006a,b), and Frangopol and Liu (2007) for an individual bridge. In this table, a larger condition index indicates an increase in the bridge deterioration (Liu and Frangopol 2004). Furuta et al. (2006) proposed a probabilistic approach for optimal bridge maintenance planning associated with bridge safety and service life. Okasha and Frangopol (2009) developed an efficient multiobjective optimization of structural maintenance considering system reliability, redundancy, and life-cycle cost. Orcesi and Frangopol (2011) studied optimum bridge maintenance planning based on SHM information, where the objectives are to minimize life-cycle maintenance and failure costs. Bocchini and Frangopol (2011a) and Frangopol and Bocchini (2012) proposed a bridge network maintenance framework that maximizes network performance and minimizes the life-cycle maintenance cost. A probabilistic approach for optimum inspection and/or monitoring planning can be found in Kim and Frangopol (2011a–c), where bi-objective optimization problems were formulated using the availability of monitoring data, the expected damage detection delay, and the inspection and/or monitoring cost.

18.5 Conclusions

Using probabilistic concepts, the life-cycle performance and cost of deteriorating bridges can be predicted in a rational way. Optimum inspection and maintenance planning can be achieved through single or multiobjective optimization. To formulate a realistic life-cycle optimization problem, the effects of inspection, monitoring, and maintenance on structural performance should be considered under uncertainty.

For practical use of life-cycle performance and cost analysis, several significant efforts are required such as (1) development of effective and practical methods to represent the structural performance including redundancy and robustness (Frangopol 2011); (2) development of more accurate prediction models of structural performance using information obtained from inspection and SHM; (3) improvement in life-cycle cost assessment and performance prediction considering extreme events such as earthquakes, floods, and hurricanes; and (4) development of life-cycle maintenance management at the

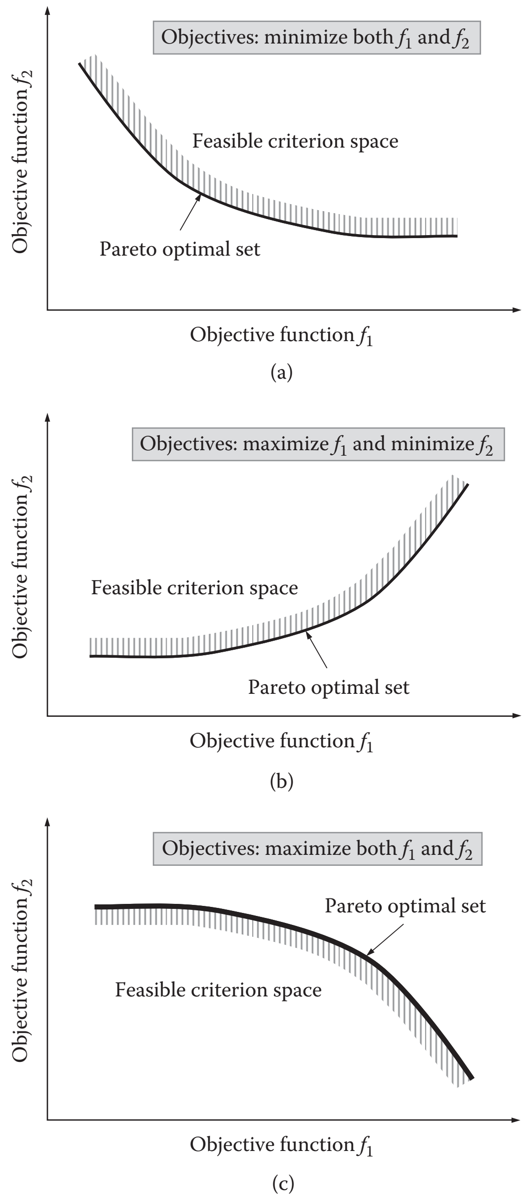


FIGURE 18.17 Pareto optimal set: (a) minimization of f_1 and f_2, (b) maximization of f_1 and minimization of f_2, and (c) maximization of f_1 and f_2.

bridge network level. Several recent studies have addressed part of these concerns. For instance, Okasha and Frangopol (2012) investigated the framework for integration of SHM in system performance-based life-cycle bridge management. An approach for bridge performance assessment and prediction using SHM data was introduced by Liu et al. (2009a,b). The approach for bridge network performance analysis and optimal maintenance planning considering extreme events was developed by Bocchini and Frangopol (2011a–c), Bocchini et al. (2011, 2013), Decò and Frangopol (2012), and Frangopol and Bocchini (2012).

TABLE 18.1 Summary of Recent Studies Using Multiobjective Optimization for Life-Cycle Optimization

Approach	Objectives	Design Variables	Source
Optimum maintenance plan for an individual bridge	• Minimize worst lifetime condition index • Maximize worst lifetime safety index • Minimize life-cycle maintenance cost	• Maintenance application times • Maintenances types (e.g., silane, cathodic protection, minor repair, replacement)	Liu and Frangopol (2005a,b); Frangopol and Liu (2007)
	• Minimize worst mean lifetime condition index • Maximize worst mean lifetime safety index • Minimize mean of life-cycle maintenance cost	• Maintenance application times • Maintenances types (e.g., silane and rebuild)	Neves et al. (2006a,b)
	• Maximize safety • Maximize service life • Minimize life-cycle maintenance cost	• Maintenance application times • Maintenances types (e.g., painting, restoring, desalting, cathodic protection, replacement)	Furuta et al. (2006)
	• Minimize probability of system failure • Maximize system redundancy • Minimize life-cycle maintenance cost	• Maintenance application times • Maintenances types (e.g., painting, replacement)	Okasha and Frangopol (2009)
	• Minimize failure cost • Minimize life-cycle maintenance cost	• Maintenance application times • Maintenances types (e.g., replacement)	Orcesi and Frangopol (2011)
Optimum maintenance plan for a bridge network	• Maximize network performance • Minimize life-cycle maintenance cost	• Maintenance application times on individual bridges	Bocchini and Frangopol (2011a); Frangopol and Bocchini (2012)
Optimum inspection and/or monitoring planning	• Minimize expected damage detection delay • Minimize total inspection/monitoring cost	• Inspection/monitoring application times • Quality of inspection/ monitoring duration	Kim and Frangopol (2011a,b)
	• Maximize availability of monitoring data for prediction • Minimize total monitoring cost	• Monitoring application times • Monitoring duration	Kim and Frangopol (2011c)

Acknowledgments

The support from the National Science Foundation through CMS-0639428; the Commonwealth of Pennsylvania, Department of Community and Economic Development, through the Pennsylvania Infrastructure Technology Alliance; the U.S. Federal Highway Administration Cooperative Agreement Award DTFH61-07-H-00040; and the U.S. Office of Naval Research Award Numbers N00014-08-1-0188 and N00014-12-1-0023 is gratefully acknowledged. The authors acknowledge the comments offered by

Alysson Mondoro and Samantha Sabatino, PhD students, Lehigh University. The opinions and conclusions presented in this paper are those of the authors and do not necessarily reflect the views of the sponsoring organizations.

Notation

C_{ET} = expected life-cycle cost
C_{FAIL} = expected failure cost
C_f = expected monetary loss due to structural failure
C_{INI} = initial cost
C_{INS} = inspection cost
C_M = maintenance cost
$E(X)$ = mean of random variable X
$f_{\mathbf{X}}(\mathbf{x})$ = joint probability density function of random variables **X**
$g(\mathbf{X})$ = state function consisting of random variables **X**
N_{ins} = number of inspections
$p_{branch,i}$ = occurrence probability of ith branch in event tree
$p_{ins,i}$ = probability of damage detection for ith inspection
p_F = probability of failure
$p_{F,life}$ = lifetime probability of failure
p_S = reliability
$p_{S,life}$ = lifetime reliability
$p_{S,thres}$ = threshold of reliability
P = structural performance
P_o = initial structural performance
ΔP = structural performance improvement
t_e = upper bound of damage occurrence time
t_{ini} = time of deterioration initiation
t_{ins} = inspection time
t_d = deterioration delay time
t_{delay} = damage detection delay
t_{life} = service life time
$t_{life,o}$ = initial service life time
$t_{main,i}$ = ith maintenance time
t_s = lower bound of damage occurrence
X = random variable
X' = standard normal variable associated with X
β = reliability index
β_i = reliability index of component i
β_s = reliability index of series system
β_p = reliability index of parallel system
β_{sp} = reliability index of series–parallel system
Φ = inverse of the standard normal cumulative distribution function
μ_X = mean of random variable X
σ_X = standard deviation of random variable X

References

Akgül, F. 2002. Lifetime System Reliability Prediction for Multiple Structure Types in a Bridge Network. PhD thesis, University of Colorado, Boulder, CO.

Akiyama, M., Frangopol, D.M., and Matsuzaki, H. 2011. "Life-cycle reliability of RC bridge piers under seismic and airborne chloride hazards," *Earthquake Engineering & Structural Dynamics*, 40(15), 1671–1687.

Al-Tayyib, A.-H.J. and Khan, M.S. 1988. "Corrosion rate measurements of reinforcing steel in concrete by electrochemical techniques." *ACI Materials Journal* 85(3), 172–177.

Ang, A.H.-S. and De Leon, D. 2005. "Modeling and analysis of uncertainties for risk-informed decisions in infrastructures engineering." *Structure and Infrastructure Engineering* 1(1), 19–31.

Ang, A.H.-S. and Tang, W.H. 1984. *Probability Concepts in Engineering Planning and Design*, Volume II. John Wiley & Sons, New York, NY.

Arora, J.S. 2004. *Introduction to Optimum Design*. Elsevier Academic Press, San Diego, CA.

Arora, P., Popov, B.N., Haran, B., Ramasubramanian, M., Popova, S., and White, R.E. 1997. "Corrosion initiation time of steel reinforcement in a chloride environment—A one dimensional solution." *Corrosion Science* 39(4), 739–759.

ASCE. (2009). *Report Card for America's Infrastructure*. American Society of Civil Engineers, Reston, VA.

Barone, G., Frangopol, D.M., and Soliman, M. 2013. "Optimization of life-cycle maintenance of deteriorating structures considering expected annual system failure rate and expected cumulative cost," *Journal of Structural Engineering,* ASCE (in press), doi:10.1061/(ASCE)ST.1943-541X.0000812.

Bocchini, P. and Frangopol, D.M. 2011a. "A probabilistic computational framework for bridge network optimal maintenance scheduling." *Reliability Engineering & System Safety* 96(2), 332–349.

Bocchini, P. and Frangopol, D.M. 2011b. "A stochastic computational framework for the joint transportation network fragility analysis and traffic flow distribution under extreme events." *Probabilistic Engineering Mechanics* 26(2), 182–193.

Bocchini, P. and Frangopol, D.M. 2011c. "Generalized bridge network performance analysis with correlation and time-variant reliability." *Structural Safety* 33(2), 155–164.

Bocchini, P., Frangopol, D.M., and Deodatis, G. 2011. "A random field technique for the efficiency enhancement of bridge network life-cycle analysis under uncertainty." *Engineering Structures* 33(12), 3208–3217.

Bocchini, P., Saydam, D., and Frangopol, D.M. 2013. "Efficient, accurate, and simple Markov chain model for the life-cycle analysis of bridge groups," *Structural Safety*, 40(1), 51–64.

Chaker, V., ed. 1992. *Corrosion Forms & Control for Infrastructure*. STP 1137. ASTM, Philadelphia, PA.

Das, P.C. 1999. Prioritization of bridge maintenance needs. In *Case Studies in Optimal Design and Maintenance Planning of Civil Infrastructure Systems*, D.M. Frangopol, ed. ASCE, Reston, VA, 26–44.

Decò, A., and Frangopol, D.M. 2012. "Life-cycle risk assessment of spatially distributed aging bridges under seismic and traffic hazards," *Earthquake Spectra*, 29(1), 127–153.

Dhir, R.K., Hewlett, P.C. and Chan, Y.N. 1989. "Near-surface characteristics of concrete: prediction of carbonation resistance." *Magazine of Concrete Research* 41(148), 137–143.

Enright, M.P. and Frangopol, D.M. 1999. "Maintenance planning for deteriorating concrete bridges," *Journal of Structural Engineering*, A 125(12), 1407–1414.

Estes, A.C. and Frangopol, D.M. 1998. "RELSYS: a computer program for structural system reliability analysis." *Structure Engineering & Mechanics* 6(8), 901–919.

Estes, A.C. and Frangopol, D.M. 1999. "Repair optimization of highway bridges using system reliability approach." *Journal of Structural Engineering* 125(7), 766–775.

Estes, A.C. and Frangopol, D.M. 2001. "Minimum expected cost-oriented optimal maintenance planning for deteriorating structures: application to concrete bridge decks." *Reliability Engineering & System Safety* 73(3), 281–291.

Estes, A.C. and Frangopol, D.M. 2005. Life-cycle evaluation and condition assessment of structures. In *Handbook of Structural Engineering*, Chapter 36, 2nd edition, W.-F. Chen and E.M. Lui, eds. CRC Press, Boca Raton, FL, 36–1 to 36–51.

Farhey, D.N. 2005. "Bridge instrumentation and monitoring for structural diagnostics." *Structural Health Monitoring* 4(4), 301–318.

Fisher, J.W. 1984. *Fatigue and Fracture in Steel Bridges: Case Studies*. John Wiley & Sons, New York, NY.

Frangopol, D.M. 1995. "Reliability-based optimum structural design," Chapter 16 in *Probabilistic Structural Mechanics Handbook*, C. Sundararajan, ed., Chapman & Hall, New York, 352–387

Frangopol, D.M. 2011. "Life-cycle performance, management, and optimization of structural systems under uncertainty: accomplishments and challenges." *Structure and Infrastructure Engineering* 7(6), 389–413.

Frangopol, D.M. and Bocchini, P. 2012. "Bridge network performance, maintenance, and optimization under uncertainty: accomplishments and challenges." *Structure and Infrastructure Engineering* 8(4), 341–356. doi:10.1080/15732479.2011.563089

Frangopol, D.M. Kong, J.S., and Gharaibeh, E.S. 2001. "Reliability-based life-cycle management of highway bridges." *Journal of Computing in Civil engineering* 15(1), 27–34.

Frangopol, D.M. Lin, K.Y., and Estes, A.C. 1997a. "Life-cycle cost design of deteriorating structures." *Journal of Structural Engineering* 123(10), 1390–1401.

Frangopol, D.M. Lin, K.Y., and Estes, A.C. 1997b. "Reliability of reinforced concrete girders under corrosion attack." *Journal of Structural Engineering* 123(3), 286–297.

Frangopol, D.M. and Liu, M. 2006. Life-cycle cost and performance of civil structures. In *McGraw-Hill 2006 Yearbook of Science and Technology*. McGraw-Hill, New York, 183–185.

Frangopol, D.M. and Liu, M. 2007. "Maintenance and management of civil infrastructure based on condition, safety, optimization, and life-cycle cost." *Structure and Infrastructure Engineering* 3(1), 29–41.

Frangopol, D.M. Strauss, A., and Kim, S. 2008. "Use of monitoring extreme data for the performance prediction of structures: general approach." *Engineering Structures* 30(12), 3644–3653.

Frangopol, D.M., Saydam, D., and Kim, S. 2012. "Maintenance, management, life-cycle design and performance of structures and infrastructures: A brief review," *Structure and Infrastructure Engineering* 8(1), 1–25.

Furuta, H., Kameda, T., Nakahara, K., Takahashi, Y., and Frangopol, D.M. 2006. "Optimal bridge maintenance planning using improved multi-objective genetic algorithm." *Structure and Infrastructure Engineering* 2(1), 33–41.

Furuta, H., Frangopol, D.M., and Nakatsu, K. 2011. "Life-cycle cost of civil infrastructure with emphasis on balancing structural performance and seismic risk of road network," *Structure and Infrastructure Engineering*, 7(1–2), 65–74.

Gonzalez, J.A., Andrade, C., Alonso, C., and Feliu, S. 1995. "Comparison of rates of general corrosion and maximum pitting penetration on concrete embedded steel reinforcement." *Cement and Concrete Research* 25(2), 257–264.

Huang, B.-S. and Chiu, H.-N. 1995. "The quality management of the imperfect production process under two monitoring policies." *International Journal of Quality & Reliability Management* 12(3), 19–31.

Irwin, G.R. 1958. *The Crack-Extension-Force for a Crack at a Free Surface Boundary*. NRL Report 5120, Naval Research Laboratory, Washington, DC.

Kim, S. and Frangopol, D.M. 2011a. "Inspection and monitoring planning for RC structures based on minimization of expected damage detection delay." *Probabilistic Engineering Mechanics* 26(2), 308–320.

Kim, S. and Frangopol, D.M. 2011b. "Optimum inspection planning for minimizing fatigue damage detection delay of ship hull structures," *International Journal of Fatigue* 33(3), 448–459.

Kim, S. and Frangopol, D.M. 2011c. "Cost-effective lifetime structural health monitoring based on availability." *Journal of Structural Engineering* 137(1), 22–33.

Kim, S., Frangopol, D.M., and Zhu, B. 2011. "Probabilistic optimum inspection/repair planning to extend lifetime of deteriorating RC structures." *Journal of Performance of Constructed Facilities* 25(6), 534–544. doi:10.1061/(ASCE)CF.1943-5509.0000197.

Kong, J.S. and Frangopol, D.M. 2003. "Life-cycle reliability-based maintenance cost optimization of deteriorating structures with emphasis on bridges." *Journal of Structural Engineering* 129(6), 818–828.

Kong, J.S. and Frangopol, D.M. 2004. "Cost-reliability interaction in life-cycle cost optimization of deteriorating structures." *Journal of Structural Engineering* 130(11), 1704–1712.

Kong, J.S. and Frangopol, D.M. 2005. "Probabilistic optimization of aging structures considering maintenance and failure cost." *Journal of Structural Engineering* 131(4), 600–616.

Liu. M. and Frangopol, D. M. 2004. "Optimal bridge maintenance planning based on probabilistic performance prediction." *Engineering Structures* 26(7), 991–1002.

Liu, M. and Frangopol, D.M. 2005a. "Bridge annual maintenance prioritization under uncertainty by multiobjective combinatorial optimization." *Computer-Aided Civil and Infrastructure Engineering* 20(5), 343–353.

Liu, M. and Frangopol, D.M. 2005b. "Multiobjective maintenance planning optimization for deteriorating bridges considering condition, safety, and life-cycle cost." *Journal of Structural Engineering* 131(5), 833–842.

Liu, M., Frangopol, D.M., and Kim, S. 2009a. "Bridge safety evaluation based on monitored live load effects." *Journal of Bridge Engineering* 14(4), 257–269.

Liu, M., Frangopol, D.M., and Kim, S. 2009b. "Bridge system performance assessment from structural health monitoring: a case study." *Journal of Structural Engineering* 135(6), 733–742.

Liu, P.-L., Lin, H.-Z., and Der Kiureghian A. 1989. *CALREL User Manual*. Report No. UCB/SEMM-89/18. Structural Engineering, Mechanics and Materials, Department of Civil Engineering, University of California, Berkeley, CA.

Marsh, P.S. and Frangopol, D.M. 2008. "Reinforced concrete bridge deck reliability model incorporating temporal and spatial variations of probabilistic corrosion rate sensor data." *Reliability Engineering & System Safety* 93(3), 364–409.

MathWorks Inc. 2009. *Optimization Toolbox™ 4 User's Guide*. The MathWorks Inc., Natick, MA.

Moan, T. 2005. "Reliability-based management of inspection, maintenance and repair of offshore structures." *Structure and Infrastructure Engineering* 1(1), 33–62.

Montemor, M.F., Simões, A.M.P., and Ferreira, M.G.S. 2003. "Chloride-induced corrosion on reinforcing steel: from the fundamentals to the monitoring techniques." *Cement & Concrete Composites* 25(4–5), 491–502.

NCHRP. 2005. *Concrete Bridge Deck Performance*. NCHRP Synthesis 333. Transportation Research Board, National Cooperative Highway Research Program, Washington, DC.

NCHRP. 2006. *Manual on Service Life of Corrosion-Damaged Reinforced Concrete Bridge Superstructure Elements*. NCHRP Report 558. Transportation Research Board, National Cooperative Highway Research Program, Washington, DC.

Neves, L.A.C., Frangopol, D.M., and Cruz, P.J.S. 2006a. "Probabilistic lifetime-oriented multiobjective optimization of bridge maintenance: single maintenance type." *Journal of Structural Engineering* 132(6), 991–1005.

Neves, L.A.C., Frangopol, D.M., and Petcherdchoo, A. 2006b. "Probabilistic lifetime-oriented multiobjective optimization of bridge maintenance: combination of maintenance types." *Journal of Structural Engineering* 132(11), 1821–1834.

Okasha, N.M. and Frangopol, D.M. 2009. "Lifetime-oriented multi-objective optimization of structural maintenance considering system reliability, redundancy and life-cycle cost using GA." *Structural Safety* 31(6), 460–474.

Okasha, N.M. and Frangopol, D.M. 2010a. "Novel approach for multicriteria optimization of life-cycle preventive and essential maintenance of deteriorating structures." *Journal of Structural Engineering* 136(8), 1009–1022.

Okasha, N.M. and Frangopol, D.M. 2010b. "Advanced modeling for efficient computation of life-cycle performance prediction and service-life estimation of bridges." *Journal of Computing in Civil Engineering* 24(6), 548–556.

Okasha, N.M. and Frangopol, D.M. 2011. "Computational platform for the integrated life-cycle management of highway bridges." *Engineering Structures* 33(7), 2145–2153.

Okasha, N.M. and Frangopol, D.M. 2012. "Integration of structural health monitoring in a system performance based life-cycle bridge management framework." *Structure and Infrastructure Engineering* 8(11), 999–1016.

Okasha, N.M., Frangopol, D.M., Fletcher, F.B., and Wilson, A.D. 2012. "Life-cycle cost analyses of a new steel for bridges." Technical Note, *Journal of Bridge Engineering* 17(1), 168–172.

Orcesi, A. and Frangopol, D.M. 2011. "Optimization of bridge maintenance strategies based on structural health monitoring information." *Structural Safety* 33(1), 26–41.

Papazian, J.M., Nardiello, J., Silberstein, R.P., Welsh, G., Grundy, D., Craven, C., Evans, L. et al. 2007. "Sensors for monitoring early stage fatigue cracking." *International Journal of Fatigue* 29(9–11), 1668–1680.

Paris, P.C. and Erdogan, F.A. 1963. "A critical analysis of crack propagation laws." *Journal of Basic Engineering* 85(4), 528–533.

Petcherdchoo, A., Neves, L.A.C., and Frangopol, D.M. 2008. "Optimizing lifetime condition and reliability of deteriorating structures with emphasis on bridges." *Journal of Structural Engineering* 134(4), 544–552.

Stewart, M.G. 2004. "Spatial variability of pitting corrosion and its influence on structural fragility and reliability of RC beams in flexure." *Structural Safety* 26(4), 453–470.

Stewart, M.G. and Rosowsky, D.V. 1998. "Time-dependent reliability of deteriorating reinforced concrete bridge decks." *Structural Safety* 20(1), 91–109.

Tuutti, K. 1982. *Corrosion of Steel in Concrete*. Swedish Cement and Concrete Research Institute, Stockholm, 17–21.

Val, D.V. and Melchers, R.E. 1997. "Reliability of deteriorating RC slab bridges." *Journal of Structural Engineering*, 123(12), 1638–1644.

Zhang, J. and Lounis, Z. 2006. "Sensitivity analysis of simplified diffusion-based corrosion initiation model of concrete structures exposed to chlorides." *Cement and Concrete Research* 36(7), 1312–1323.

19

Bridge Construction Methods

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19.1 Introduction

Bridge planning, design, construction, and operation are all key stages for bridge projects. This chapter discusses some examples of bridge construction methods.

Experience in bridge construction is gained by working on site. Nevertheless, it would still be useful to summarize and describe previous experiences.

A list of books, articles, and guidelines are given at the end of the chapter, presenting experiences from different countries.

The chapter is organized as follows:

Section 19.2 presents a survey of the "classification" methods for bridge construction.

It is almost impossible to present a definite classification of bridge construction methods, as there will always be new construction methods and combination of methods, depending on the project requirements.

However, it would still be useful—in particular for educational purposes—to discuss bridge classification. Section 19.2 could be considered also as a survey of some of the main references discussing bridge construction. Section 19.2 presents a proposed classification of bridge construction methods. Sections 19.3–19.14 present discussions of some of the common bridge construction techniques. Emphasis is on prestressed concrete structures.

19.2 Classifications of Bridge Construction Methods

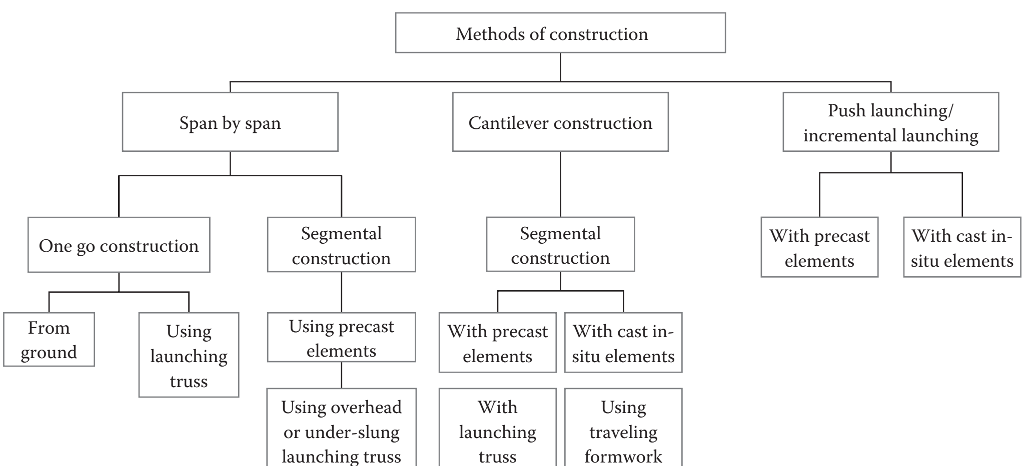
Several construction systems for bridges superstructures have been developed. This section presents a review of some of the construction methods and classification/lists of the commonly used methods. A proposed classification is presented in Section 19.2.2.

19.2.1 Review

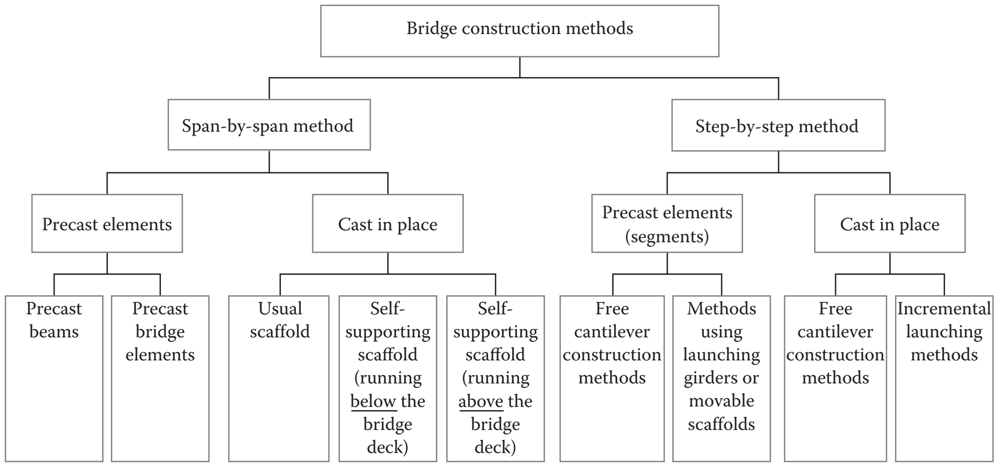
The following flow charts, tables, and figures also present a review of the various classifications of bridge construction methods as mentioned in several text books and papers.

19.2.1.1 Rajagopalan (2006) Classifications

Bridge construction methods classified by Rajagopalan (2006) are based on the methods of erection to span-by-span construction, cantilever construction, or push launch construction. Span-by-span construction could be in one go with precast girders (mostly "I" girders) or using segmental erection with precast units. Cantilever construction could be only segmental construction with precast or cast in situ segments. Push launch method could be with precast or cast in situ units moved over span by span from one end to reach the other end.



19.2.1.2 Jungwirth (1998) Classification



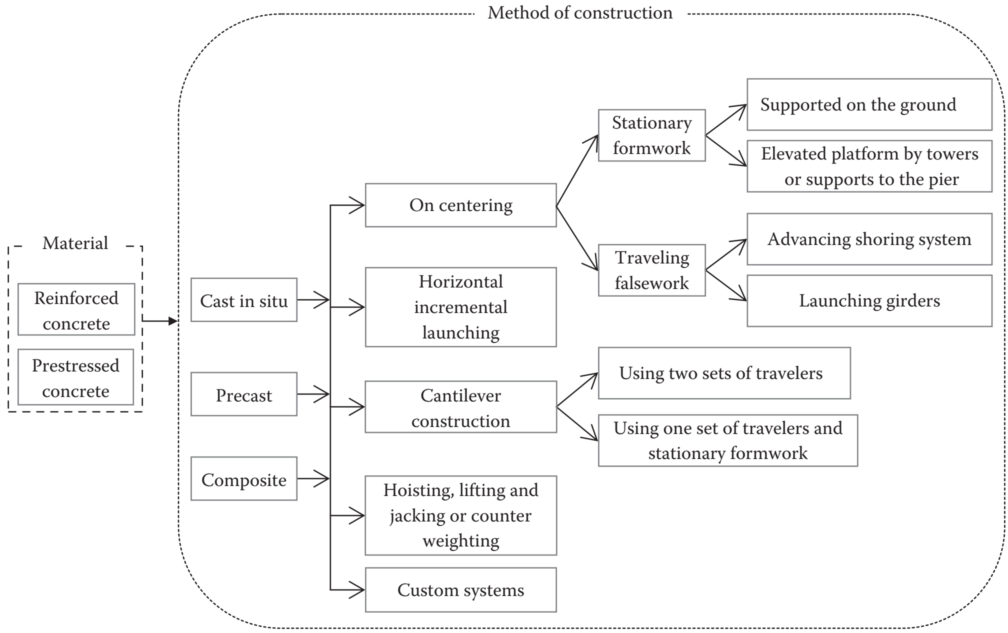
19.2.1.3 Combault (2008) Classifications

Concrete deck structures are either cast in place (i.e., at their final location) or built in a practical place before being moved to their final location, or prefabricated in many pieces then transported and assembled at their final location. This leads to classify the construction methods as follows Combault (2008).

- Cast-in-situ-methods
  - Forms on scaffolding
  - Forms on temporary supports
  - Self-supported forms
    - Span-by-span
    - Balanced cantilever erection
- Displacement methods
  - Translation
    - Longitudinal translation (incremental launch)
    - Transversal translation
    - Vertical translation
  - Rotation
    - Around a vertical axis
    - Around a horizontal axis
- Prefabrication
  - Beams placed with cranes or launching gantries
  - Box girders
    - Short line
    - Long line
      - Balanced cantilever erection
      - Progressive placing
      - Span-by-span construction

19.2.1.4 Youssef-Anumba-Thorpe (2005) Classifications

Bridge superstructure construction can be classified according to material and method of erection (Youssef, Anumba and Thorpe 2005). A concrete bridge may be constructed using either reinforced or prestressed concrete. The method of erection can be divided into four main techniques: on centering, horizontal incremental launching, cantilever construction, and lifting or jacking (Liebenberg, 1992). However, it is important to include another category (i.e., custom systems). Some of these techniques are carried out using cast-in-situ concrete while others use precast or a combination of both. The classification of construction methods is presented as follows (Liebenberg, 1992):



19.2.1.5 Basha (1991) Classifications for Egypt

There are eight main construction methods used in Egypt: precast concrete girders, incremental launching, prefabricated steel construction, cast-in-place free cantilever, precast segmental free cantilever, precast segmental on falsework, cast-in-place reinforced concrete on falsework, and cast-in-place prestressed concrete on falsework.

19.2.1.6 General Comments on Methods of Construction (CEB-FIB 2000)

1. In-situ construction on scaffolding or stationary falsework
    1.1 Classical scaffolding
    1.2 Stationary falsework
2. Launching girder or gantry
    2.1 Launching gantries
    2.2 Launching girders
3. Cantilever construction
    3.1 In-situ construction
    3.2 The use of precast elements
4. Incremental launching
    4.1 Launching nose
    4.2 Temporary supports
    4.3 Launching cable-stays
5. Other methods of movement
    5.1 Rotation
    5.2 Transverse movement
6. Precast construction
7. Transportation and lifting of heavy loads

Source: fib CEB-FIP Bulletin No. 9 (2000), Part II, Section 4.

19.2.1.7 Construction of Girder Bridges (Benaim 2008)

1. Cast-in-situ span-by-span construction of continuous beams
2. Precast segmental span-by-span erection
3. Cast-in-situ balanced cantilever construction
4. Precast segmental balanced cantilever construction
5. Progressive erection of precast segmental decks
6. Incremental launching
7. Prefabrication of complete spans

Source: Benaim, R., *The Design of Prestressed Concrete Bridges: Concepts and Principles*, CRC Press, Boca Raton, FL, 2008, Chapter 15.

19.2.1.8 Concrete Bridge Superstructure Construction (Liebenberg 1983)

There are three main methods of construction for concrete, which are as follows:

1. In situ casting in formwork in position on the works.
2. Precasting off the works and subsequent transportation and erection.
3. Composite, being a combination of the preceding two points.

The four main forms of erection are as follows:

1. On centering, that is, stationary falsework supported directly at ground level or in the form of fixed girders or arches, or traveling falsework supported on the substructure or, when necessary, also on intermediate towers.
2. By cantilevering from previous sections or substructures with or without suspended cable support.
3. Horizontal incremental jacking.
4. By vertical hoisting, lifting, or jacking.

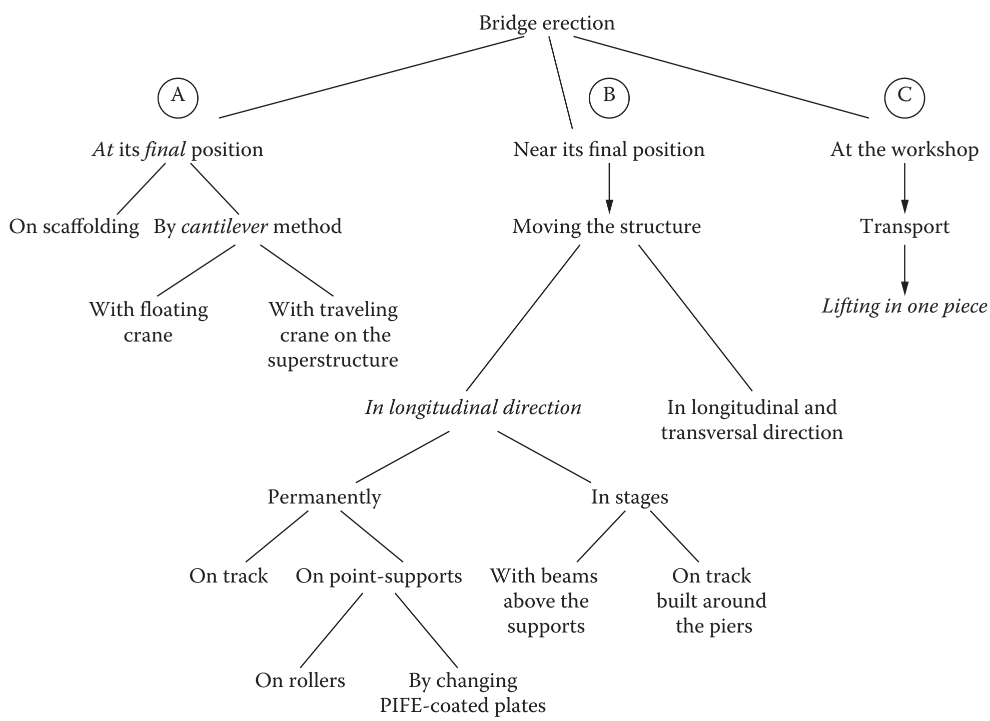
19.2.1.9 Classification of Erecting Method (JICA 1991)

Classification (I)	Classification (II)	Classification (III)
Construction method of cast-in-place and precast block beam on false work	1. Frame scaffolding 2. Beam scaffolding 3. Beam and shoring scaffolding	
Erecting method of precast PC beam	Erecting method by steel girder	1. Erecting method by two steel girders 2. Erecting method by hanging from one steel girder 3. Erecting method by using one steel girder and two portal cranes 4. Shift erecting by one steel girder 5. Shift erecting by two steel girders 6. Carrying erection method
	Erecting method by crane	1. Erecting by a single truck crane 2. Erecting by two truck cranes 3. Erecting by floating crane 4. Tower erecting method
	Erecting method by gantry crane	1. Fixed gantry crane erecting method 2. Self-traveling gantry crane method
	Erecting method by bent	1. Bent-type erecting 2. Traveling bent systems
Cantilever construction method	Cantilever construction method by cast-in-place	1. Cantilever construction method by form traveler (D.W.) 2. Cantilever construction method by movable truss (P&Z)

(*Continued*)

Classification (I)	Classification (II)	Classification (III)
	Cantilever erection method by precast segment	1. Cantilever erection method by erection nose 2. Cantilever erection method by erection girder 3. Cantilever erection method by erection tower 4. Cantilever erection method by cable crane 5. Cantilever erection method by gantry crane 6. Cantilever erection method by truck crane 7. Cantilever erection method by floating crane
Incremental launching method	1. Incremental launching method by one action 2. Incremental launching method by multiaction	
Movable scaffolding method	1. Movable scaffolding with framed support 2. Movable scaffolding with girder and support 3. Movable scaffolding with girder and hanger	

19.2.1.10 Typical Methods of Steel Bridge Erection in Japan



19.2.2 Proposed Classifications of Bridge Construction Methods

This section presents a proposed classification of the construction methods for bridges as shown in Figure 19.1.

Bridges can be categorized, according to the construction procedure, into three main classifications: cast in place, precast or prefabricated, and finally movement methods (see Table 19.1). Figure 19.2 gives the main construction methods with diagrams explaining the different construction procedures that can be adopted within each method.

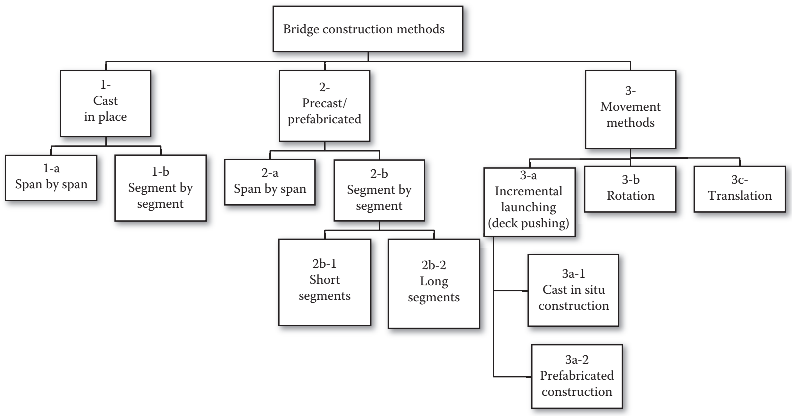


FIGURE 19.1 Proposed classification of bridge construction methods.

TABLE 19.1 Comparison between the Three Main Categories of Bridge Construction Methods

	Cast in Place	Precast/Prefabricated	Movement Methods
Bridge deck	Cast in place	Fabrication yard far away from the bridge	Fabrication yard on bridge site
Falsework	Moves from span to span	Does not move, stays on yard	Does not move
Formwork or bridge equipment	Moves	Moves	Does not move
Bridge elements during construction	Does not move	Moves	Moves

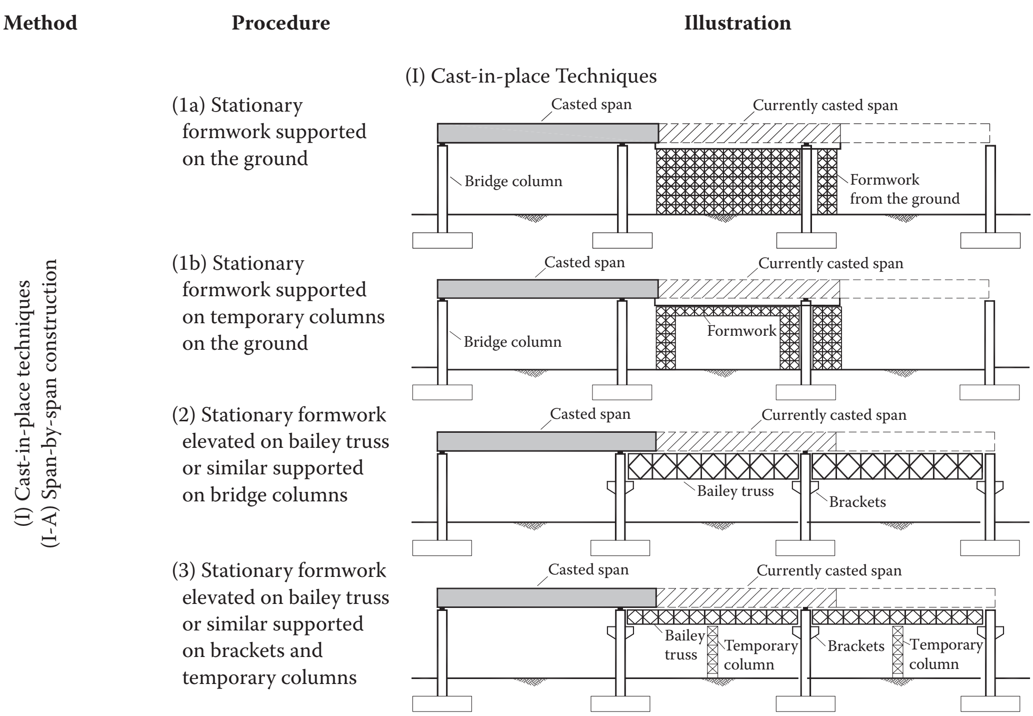
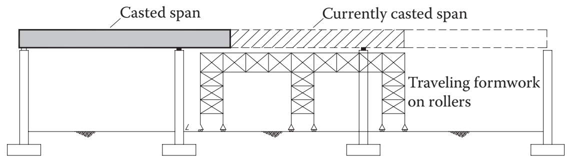


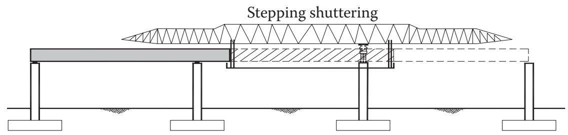
FIGURE 19.2 Different construction methods for bridges.

Method **Procedure** **Illustration**

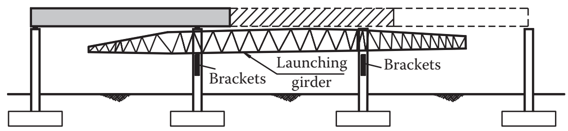
(4) Traveling formwork on rollers



(5a) Traveling formwork above the deck (movable scaffolding or flying shuttering)

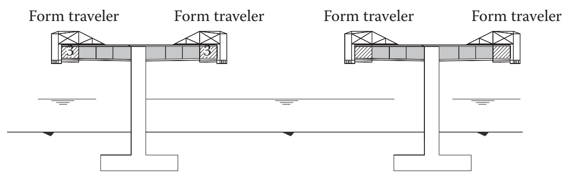


(5b) Traveling form below the deck

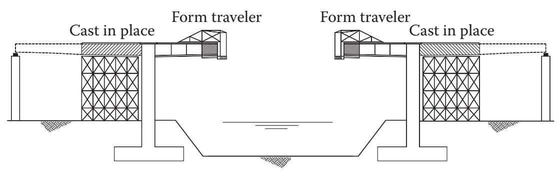


(I) Cast-in-place techniques
(I-B) Segment-by-segment construction

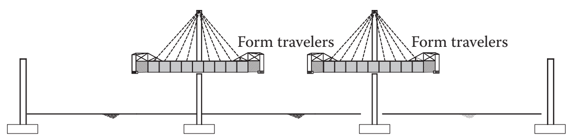
(6a) Balanced cantilever construction using two form travelers (cast-in-place technique)



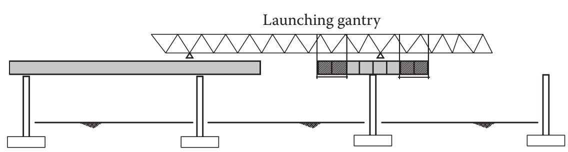
(6b) Balanced cantilever construction using one form traveler and stationary system (cast-in-place technique)



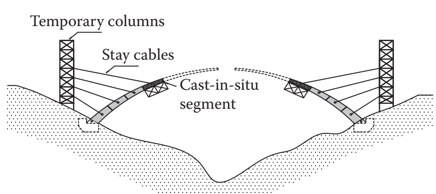
(6c) Cantilever construction using auxiliary cables



(6d) Cantilever construction using launching gantry



(6e) Arch construction using temporary columns and stay cables



(6f) Construction of cable stayed bridges using form travelers

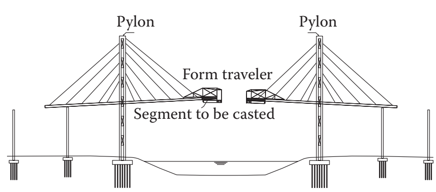


FIGURE 19.2 (*Continued*) Different construction methods for bridges.

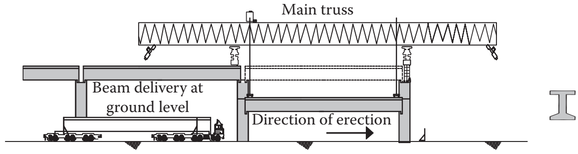
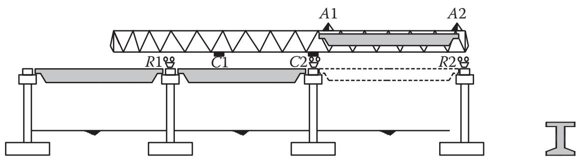
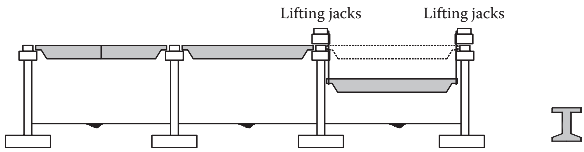
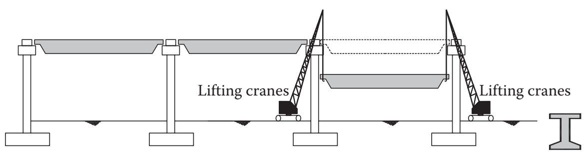
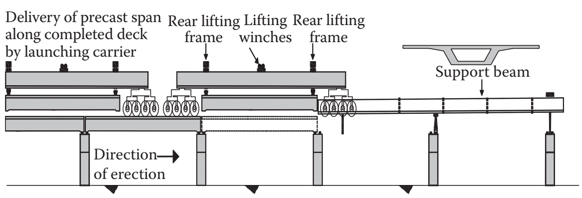
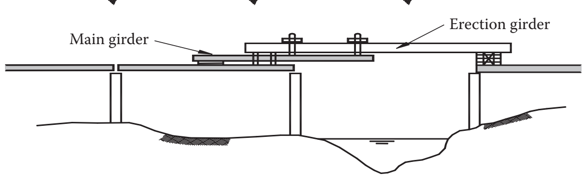
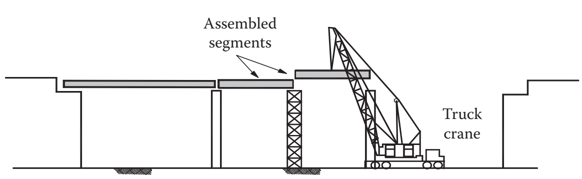
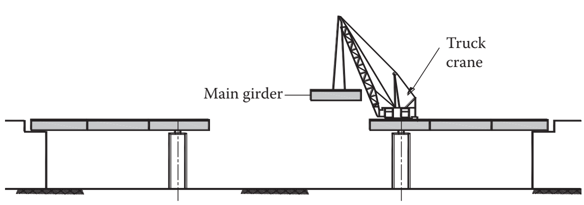
Method	Procedure	Illustration
	(II) Precast/Prefabricated Techniques	
(II) Precast/Prefabricated techniques (II-A) Span-by-span construction	(7a) Launching truss method (erection of precast beams lifted from the ground)	
	(7b) Launching truss method (erection of precast beams delivered from the deck)	
	(8a) Jacking of precast concrete beams from the ground	
	(8b) Lifting precast concrete or steel beams from the ground using cranes	
	(9) Precast full width span construction using mobile carriers (from the previously constructed deck)	
	(10) Span-by-span construction using erection girders	
(II) Precast/Prefabricated techniques (II-B) Long segment construction	(11a) Span-by-span construction by assembling the segments (half span length) using post tensioning on temporary columns	
	(11b) Construction of precast spans by lifting segments (any length) using crane placed on pre-constructed deck then assembling the segments by post tensioning	

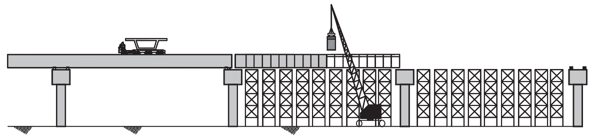
FIGURE 19.2 (***Continued***) Different construction methods for bridges.

Method **Procedure** **Illustration**

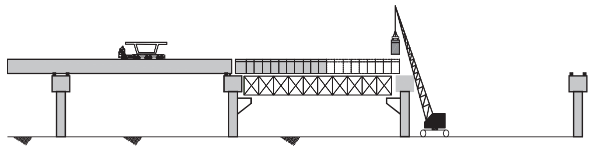
(II) Precast/Prefabricated Techniques

(II) Precast/Prefabricated techniques
(II-C) Short segment construction

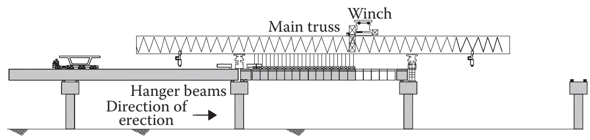
(12) Precast segmental construction on false work (segments lifted from the ground or the deck)



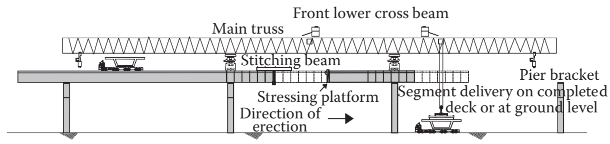
(13) Precast segmental construction on bailey truss or similar (segments lifted from the ground or the deck)



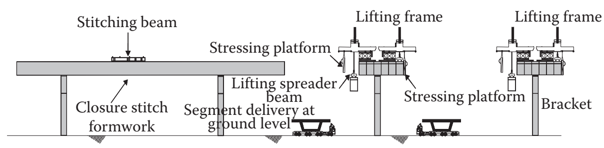
(14) Precast segmental with launching gantry (segments lifted from the deck)



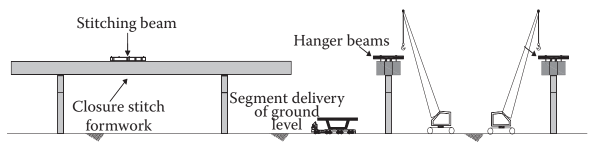
(15) Balanced cantilever construction using precast segments lifted from the ground



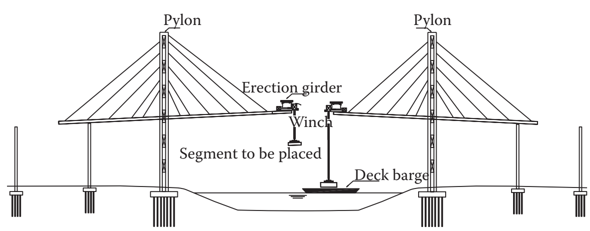
(16) Balanced cantilever construction using lifting jacks (segments lifted from the ground)



(17) Balanced cantilever construction using cranes (segments lifted from the ground)



(18a) Construction of cable stayed bridges by either lifting precast segments from barge or segments are delivered from the deck



(18b) Arch construction using erection towers and stay cables

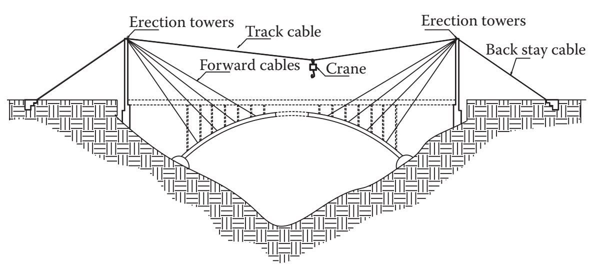


FIGURE 19.2 (***Continued***) Different construction methods for bridges.

Method	Procedure	Illustration

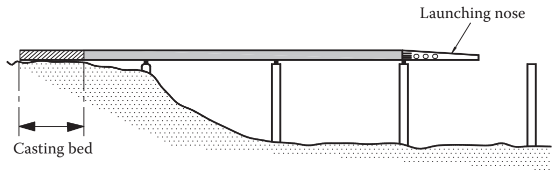
(III) Bridges constructed using movement of whole bridge

(III) Bridges constructed using movement methods

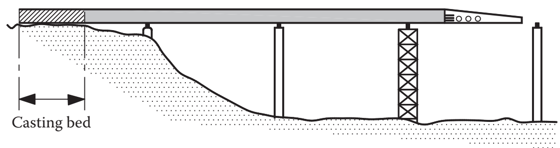
(III-A) Incremental launching (Deck pushing)

1-Cast in situ

(19a) Horizontal incremental launching using steel nose (deck pushing system)

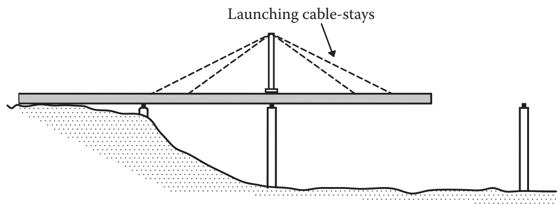


(19b) Horizontal incremental launching for medium spans using temporary support

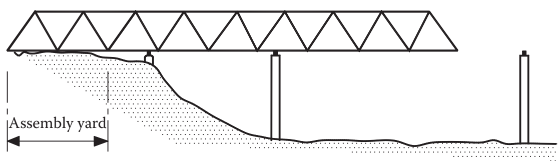


2-Prefabricated

(20a) Horizontal incremental launching (deck pushing) for long spans using launching cables

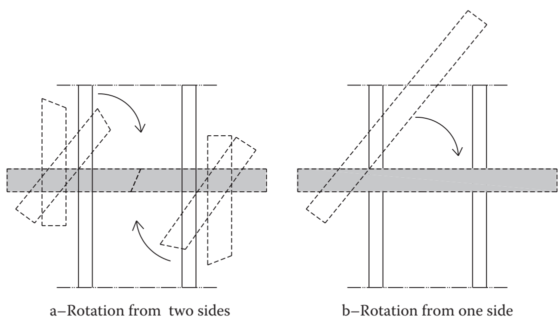


(20b) Horizontal incremental launching (deck pushing) for long spans or railway truss bridges



3b-Rotation

(20c) Rotated Bridges



3c-Translation

(20d) Transverse movement of bridges (Replacement of an old bridge or construction by lateral sliding)

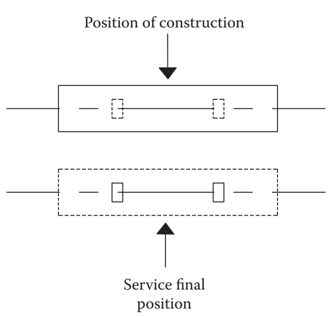
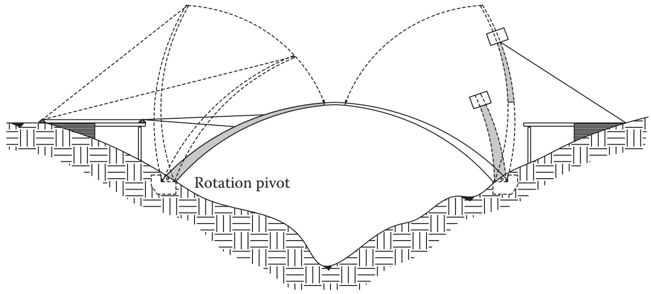


FIGURE 19.2 (***Continued***) Different construction methods for bridges.

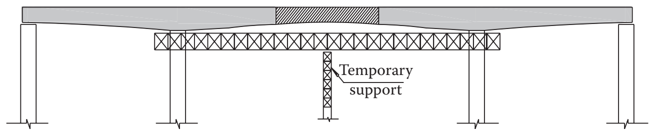
Method	**Procedure**	**Illustration**

(IV) Other special and combined methods

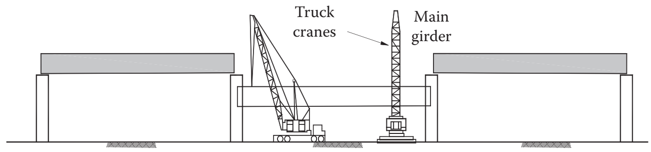
(21) Arch construction in vertical position then rotation to final position



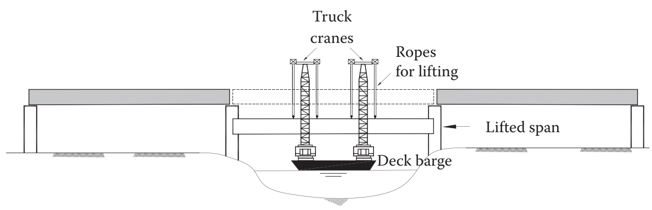
(22) Construction of central portion using movable scaffolding



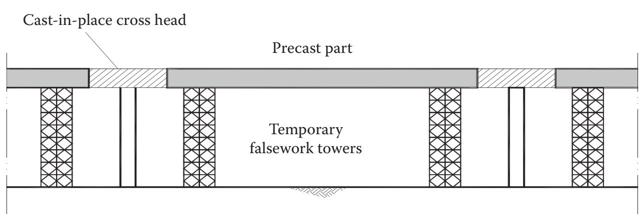
(23) Heavy lifting construction (whole span is lifted from the ground and placed at final position)



(24) Heavy lifting construction (whole span is lifted from the water and placed at final position)



(25) Construction of continuous bridges using precast beams and cast in situ cross heads



(26) Cantilever construction for truss bridges

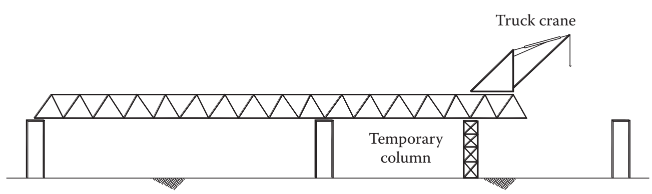


FIGURE 19.2 (*Continued*) Different construction methods for bridges.

19.3 Stationary Systems from the Ground

19.3.1 General

This system normally requires full occupancy of the ground. It is the most commonly used method in Egypt and throughout the world. In the early days, timber was the main material used in construction. This method can accommodate any deck curvature and most cross-sectional shapes, including box section, slabs, and beams. It can be used for any type of concrete and fits well with complicated configurations. It is preferred for moderate aboveground heights of around 10 m. The technique requires full accessibility to the ground to allow system erection and dismantling. There should not be any obstacles on the ground to hinder system erection, and the ground should be able to sustain loads transmitted by the falsework and should be without much variation in level. Cranes are generally used to help during erection. This method may not be preferred in crowded areas.

19.3.2 Construction Sequence

First, the ground should be prepared to accommodate loads, usually using concrete blinding or by simply placing timber under the shore brace legs. Some clearing, grubbing, and leveling may be required. Falsework is normally constructed in small towers constituted by the proprietary system elements and connected together using pipes, connectors, and cross bracings. Normally the final element in the falsework is used to adjust the levels of the formwork. The levels should be adjusted to cater for the deflections that will occur due to the soil and falsework (i.e., by creating camber). Normally timber is placed over the shore brace elements to provide support for the formwork. After erecting the formwork, steel reinforcement and prestressed cables are fixed. In the case of box sections, which are commonly used in Egypt, placing concrete is normally performed in two stages: the bottom slab and webs are first cast, then top the slab. After striking the inner formwork for webs, the falsework and formwork for the top slab are erected and steel reinforcement and prestressed cables (if used) are fixed before placing the concrete. After the concrete gains the required strength, the system is dismantled and transported to another span and the same cycle is repeated, as illustrated in Figure 19.3.

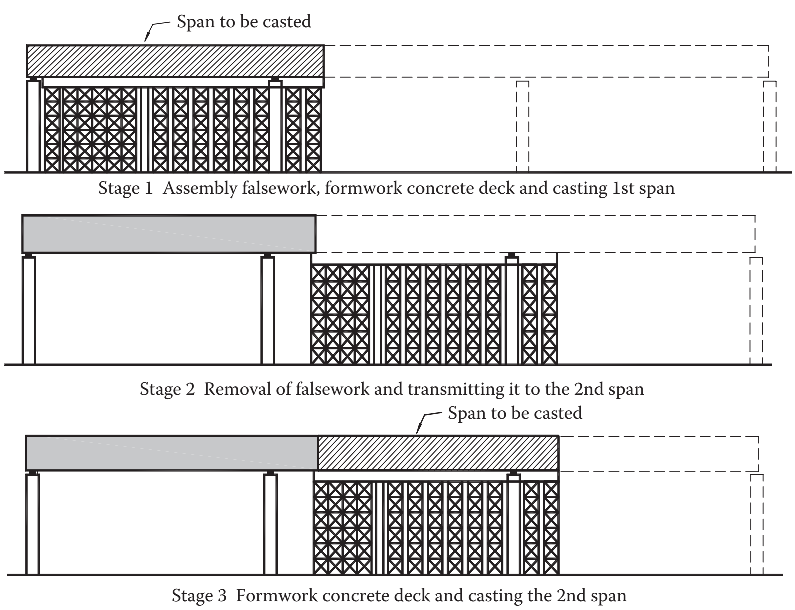


FIGURE 19.3 Construction stages using stationary formwork.

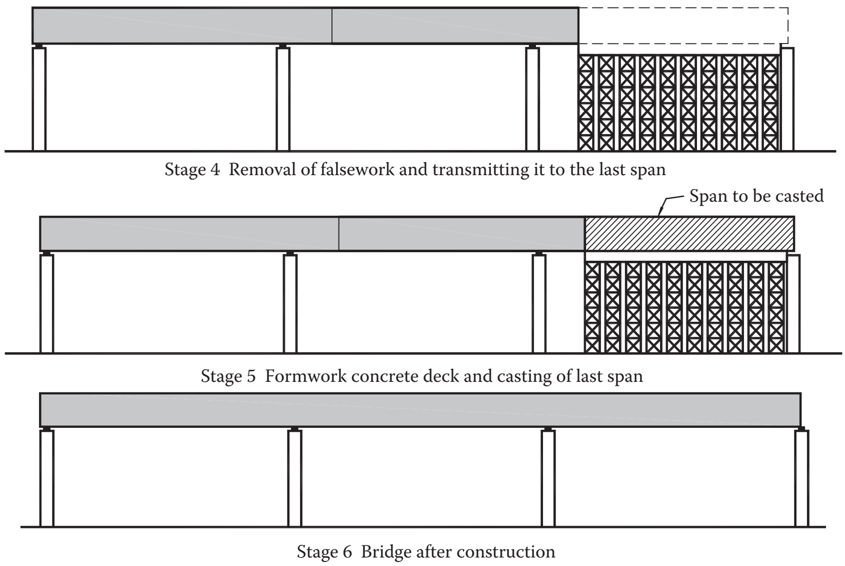


FIGURE 19.3 (*Continued*) Construction stages using stationary formwork.

19.4 Stationary System Using an Elevated Platform (Bailey Truss)

19.4.1 General

In this method, an elevated platform is installed and supported either on Bailey towers, with limited occupancy of the ground, or on brackets attached to bridge piers. The platform may be totally assembled on the ground and then lifted into position or assembled in position. The main components used in this method are pier brackets or Bailey units, steel beams, falsework, and formwork. This method is used for any deck cross section and for any curvature. It can be used for any span length provided that intermediate Bailey towers are provided every 20–25 m. It can be used for heights up to 25 m and for any type of concrete.

Execution requires access to allow system erection and dismantling. It can be used to cross obstacles or for limited widths such as railway lines, roads, and small canals. Ground conditions should be able to sustain loads transmitted by Bailey units, otherwise temporary footings may be required. It can accommodate variations in the ground topography. This method generally results in less interference with the public compared with the previous one. Also, no considerable loads affecting the permanent structure would result, except in the case of brackets. Execution requires skilled laborers for assembly and erection of the framing on the ground. The low cost of the system enables most contractors to adopt it in different projects, although brackets may be expensive due to the cost of manufacturing. The system components are generally durable and suitable for reuse several times. The time required for construction is less compared with the previous method.

19.4.2 Construction Sequence

The construction sequence depends on the method used. In the case where Bailey panels are used to have an elevated platform, the units are assembled in a horizontal or vertical fashion, as required. There are special pins and angles to connect panels together, and the arrangement of the assembled panels depends on the straining actions resulting from loads transmitted by the falsework and formwork. Figure 19.4 shows steel beams used as shuttering supported on vertical Bailey trusses set as shoring supports. Figure 19.5 shows a Bailey truss supported on brackets and temporary support at the middle.

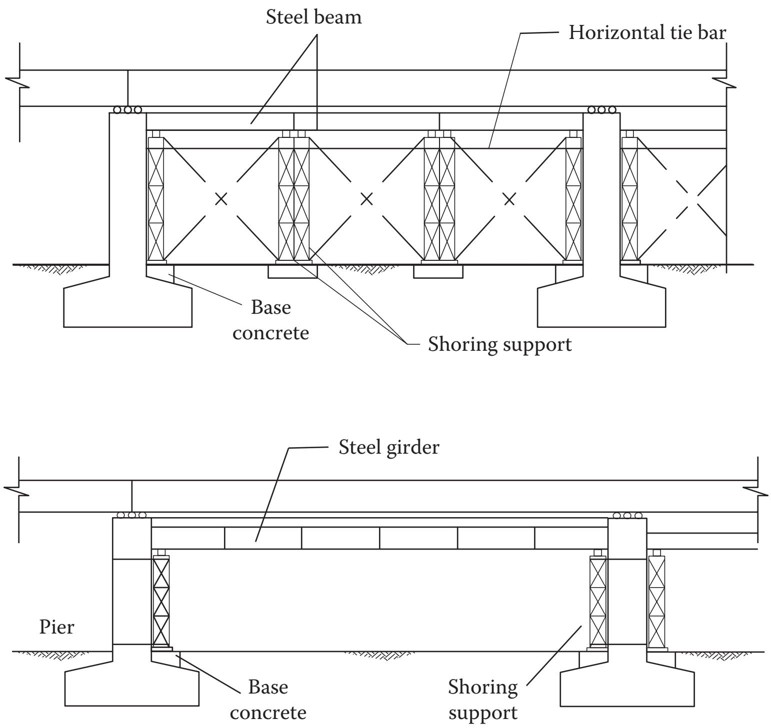


FIGURE 19.4 Steel beams (falsework) supported on vertical bailey trusses.

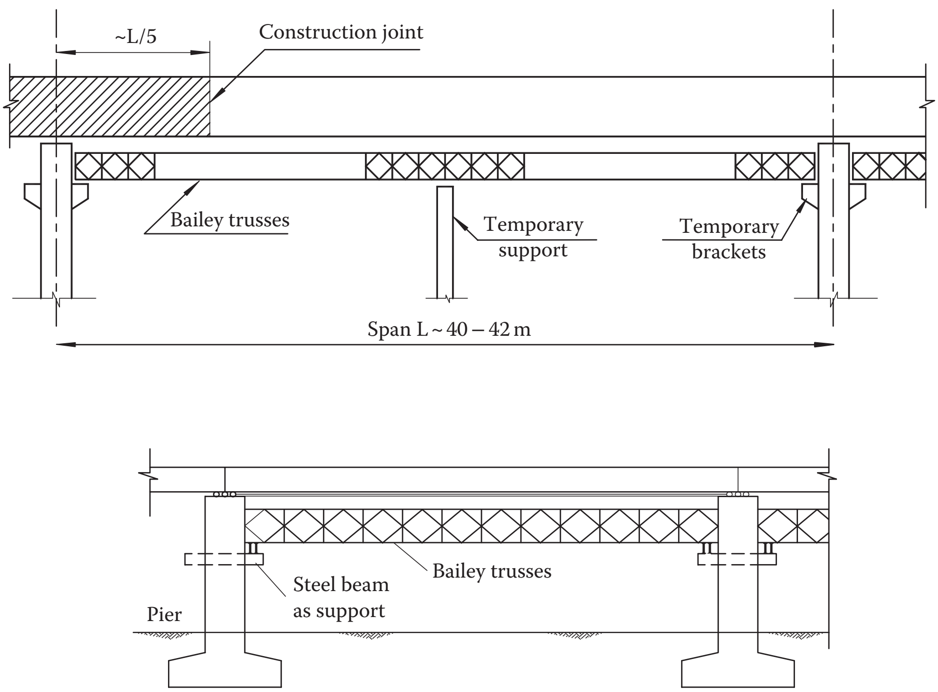


FIGURE 19.5 Bailey trusses supported on brackets installed to columns or piers.

19.5 Movable Scaffolding (Traveling Formwork, Flying Shuttering)

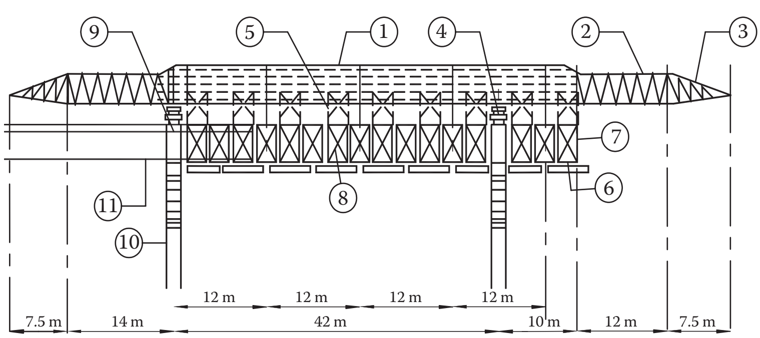
19.5.1 General

This section describes the utilization of the movable scaffolding system in Egypt through its application in four bridges constructed in Egypt. This system was adopted during the construction of the 6th of October Bridge, the eastern and western approaches of the Suez Canal Bridge, and the Luxor Bridge. The system can be adopted for a box section (which is the common use in Egypt) and for beam and slab cross sections as well. It can be used for any height, provided that there is enough space for system components available, except in the case of using cranes to erect brackets at each span.

The superstructure can be constructed regardless of the ground conditions, so obstacles have little effect on construction except during erection and dismantling, where cranes are required. Execution is mostly done on the superstructure with little interference with the public. The system may be reused after modification for similar projects. Bridge design should take into account the effect of brackets on piers and the effect of supporting the whole system on the superstructure. The system is considered the most appropriate for concrete bridges of moderate spans ranging from 40 to 70 m. The movable shuttering system provides appropriate means to satisfy local needs for concrete bridge construction in an urban environment, practically without infringing on traffic or the property below. Section 19.5.2 illustrates how this method of construction was used in Egypt for the first time to build the 6th of October Bridge extension.

19.5.2 Construction of 6th of October Bridge Extension

This system is progressive and is supported on brackets fixed to the bridge piers. The main components of the system are two main trusses, either above or below the superstructure, two brackets, and a system of formwork, as illustrated in Figure 19.6. The length, span, and width of the bridge under study are 1470, 42, and 18.4, respectively. The bridge design is a posttensioned box girder type with a top slab width of 18.4 m, web height of 2.45 m, and a bottom slab width of 10 m. The bridge was constructed in an urban area, with a railway passing near the bridge. Under these circumstances, a construction system that is capable of building the bridge without infringing on traffic or the property below was essentially needed.



No.	Item
1	Main girder
2	Launching truss
3	Launching nose
4	Support frame
5	Hanger frame
6	Base slab formwork
7	Outer formwork
8	Inner formwork
9	Hydraulic jacks
10	Piers
11	Bridge box girder

FIGURE 19.6 General view of movable scaffolding and its components.

FIGURE 19.7 Stepping shuttering during concreting phase (6th October Bridge).

FIGURE 19.8 Stepping shuttering during advancing phase (6th October Bridge).

Section 19.5.2.1 describes how this system was first adopted during construction of the 6th of October Bridge extension. The construction sequence and the main components of the stepping shuttering are also illustrated. The construction of concrete bridges using stepping shuttering is performed through two major phases: concreting and advancing phases. Figures 19.7 and 19.8 show the stepping shuttering system during the concreting and advancing phases, respectively.

19.5.2.1 Construction Sequence

The main steps for both concreting and advancing phases are as follows:

Step 1: The shuttering was assembled on an existing bridge box girder constructed with the traditional formwork.

Step 2: The shuttering was hydraulically pushed until supported on the existing bridge box girder and Pier A, as shown in Stage 1 of Figure 19.9.

Step 3: The concreting phase was started by casting the bottom slab of the bridge box girder.

Step 4: The bottom slab was cured for 24 h and then a formwork was laid down on the bottom slab to cast the web and the top slab of the bridge box girder.

Step 5: The webs and the top slab were cast and then the prestressing operation was carried out using a prestressing jack of 50 tons with a stroke of 100 mm. The prestressing process was started 3 days after casting the top slab and webs. It was carried out for the whole box girder at the same time.

Step 6: The stepping shuttering was opened and advanced using groups of hydraulic jacks. The advancing process was performed in several increments until the system was supported on Piers A and B, as shown in Stage 2 of Figure 19.9.

Step 7: The stepping shuttering was prepared to cast the next bridge span, and so on.

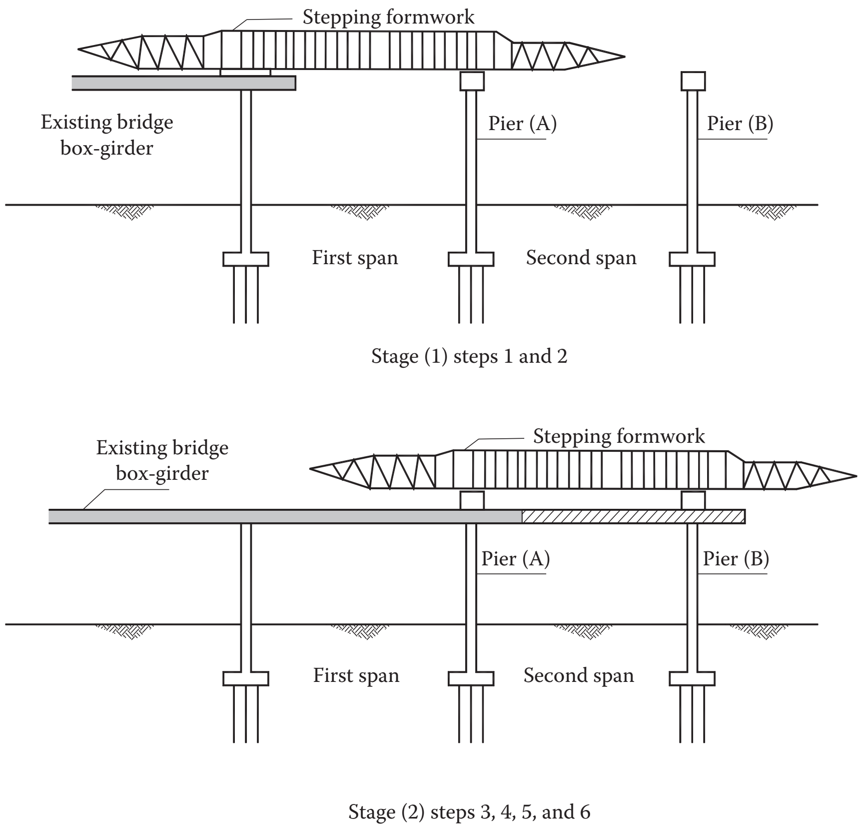


FIGURE 19.9 Stages of systematic construction.

19.5.3 Construction of Luxor Bridge Crossing the Nile

One of the best applications of the traveling scaffolding is discussed in this section. This method is traditionally applied in the construction of the approaches of bridges spanning from 25 to 45 m in most cases. One new application of this method was presented during the construction of Luxor Bridge crossing the Nile, where this method was used in the construction of the bridge approaches with spans of 40 m and was also adopted during the construction of the main navigational bays crossing the Nile with spans of 47, 90, and 47 m. Commonly the navigational bays in bridges crossing rivers are constructed using the balanced cantilever method, but in this case, the main part was constructed with the same flying shuttering that was adopted in the construction of the approaches, saving time and the expense of the utilizing the cantilever method or other methods requiring special labor and automations.

The bridge consists of 14 repetitive bays of spans 40 m and three navigational bays, two of spans 47 m and one central bay of span 90 m. The clear height of the navigational bays was 13 m. The total bridge length was 744 m and width was 21.5 m. The bridge cross section consists of two lanes in each direction with width 7.5 m and two sidewalks for pedestrians with widths of 2.65 m each. The middle median is 1.2 m wide. Figure 19.10 shows the Luxor Bridge's configuration.

19.5.3.1 Construction Sequence

The construction of the main part of the bridge, comprising the three navigational bays of spans 47, 90, and 47, respectively, was carried out using the traveling shuttering technique. The construction stages can be summarized in seven stages. The first three stages are concerned with the east part of the bridge, the next three stages for the west part, and the seventh stage for the closure segment construction.

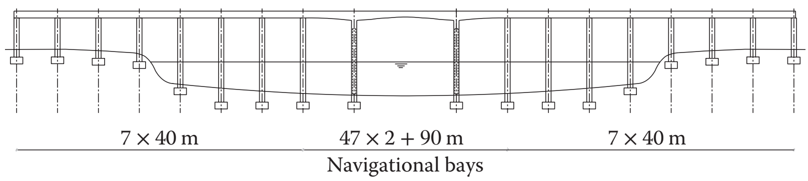


FIGURE 19.10 Luxor Bridge layout.

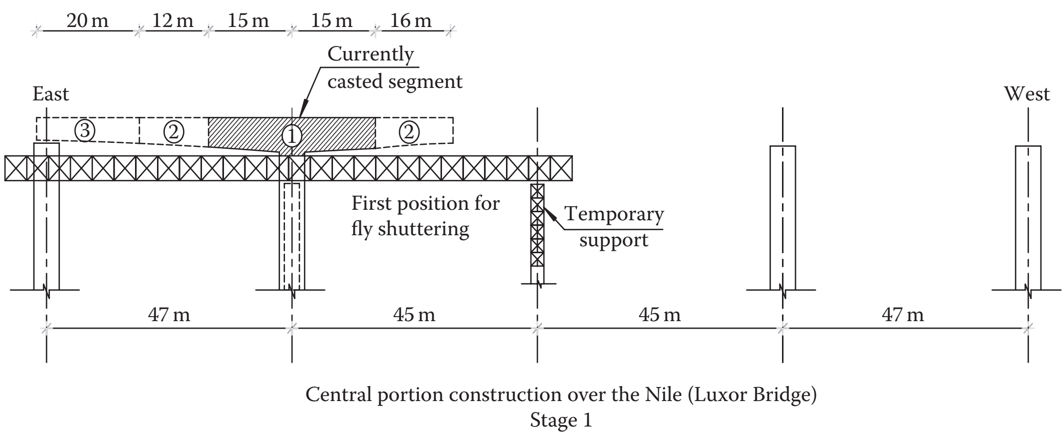


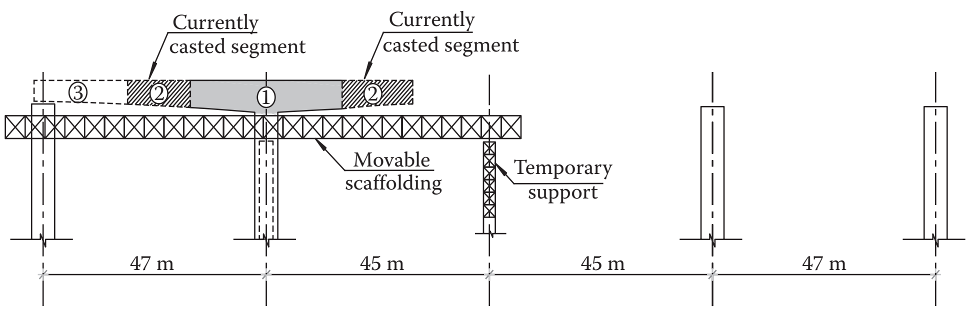
FIGURE 19.11 Casting the first segment after placement of the shuttering.

Stage 1: Placing the shuttering in the east part of the bridge and casting the first segment with length 30 m. This casted segment is shaded as shown in Figure 19.11.
Stage 2: Casting the second segment to the right and to the left of the previously casted segment, as shown in Figure 19.12.
Stage 3: Casting the terminal segment at the end to finalize the construction of the eastern portion of the bridge, as shown in Figure 19.13.
Stage 4: Casting the fourth segment after moving the shuttering to the new position at the west of the bridge, as shown in Figure 19.14.
Stage 5: Casting the fifth segment to the right and to the left of the previously casted segment, as shown in Figure 19.15.
Stage 6: Casting the terminal segment at the end to finalize the construction of the western portion of the bridge, as shown in Figure 19.16.
Stage 7: Casting the closure segment of the bridge after moving the shuttering to its new position, as shown in Figure 19.17.

Figure 19.18 shows the bridge in the final stage after construction and removal of the shuttering, while Figure 19.19 shows the movable scaffolding adopted during the construction of Luxor Bridge over the Nile.

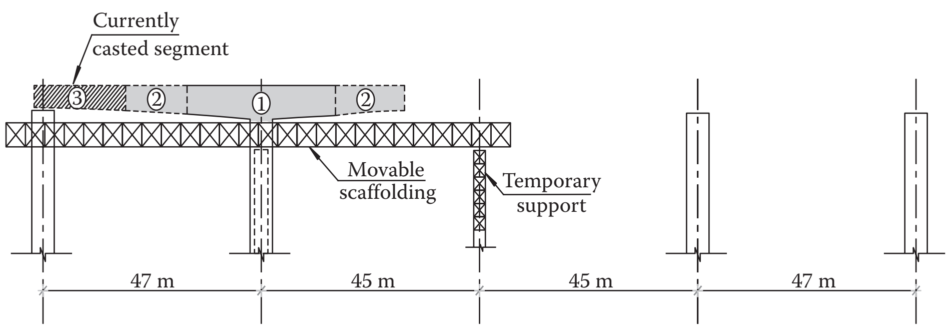
19.5.4 Construction of Suez Canal Bridge Approaches

Two Egyptian contractors began the construction of the Suez Canal Bridge in June 1997 for the west and east portions. The construction of the central portion started in May 1998 and consists of the main bridge across the Suez Canal and the approach bridges 49.5 m high. The supervisory services for the entire project and the construction of the central portion were financed by the Japanese Grant Aid.



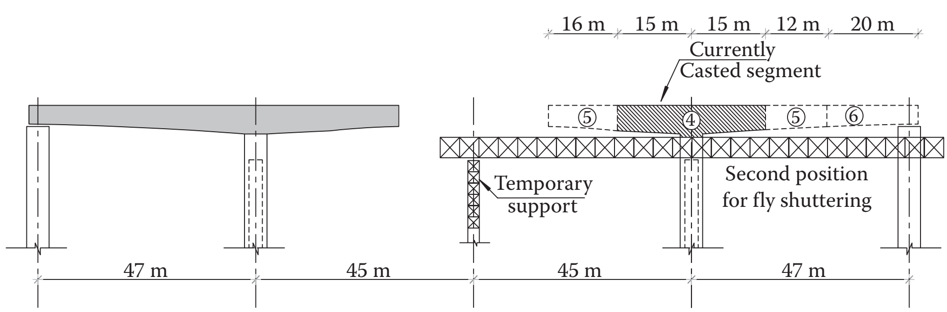
Central portion construction over the Nile (Luxor Bridge)
Stage 2

FIGURE 19.12 Casting the second segments.



Central portion construction over the Nile (Luxor Bridge)
Stage 3

FIGURE 19.13 Casting the terminal segment at the end.

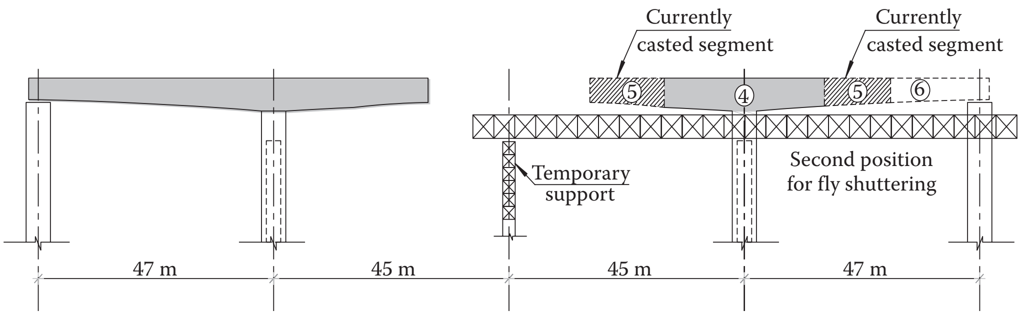


Central portion construction over the Nile (Luxor Bridge)
Stage 4

FIGURE 19.14 Casting the fourth segment after moving the shuttering to the new position (west of the bridge).

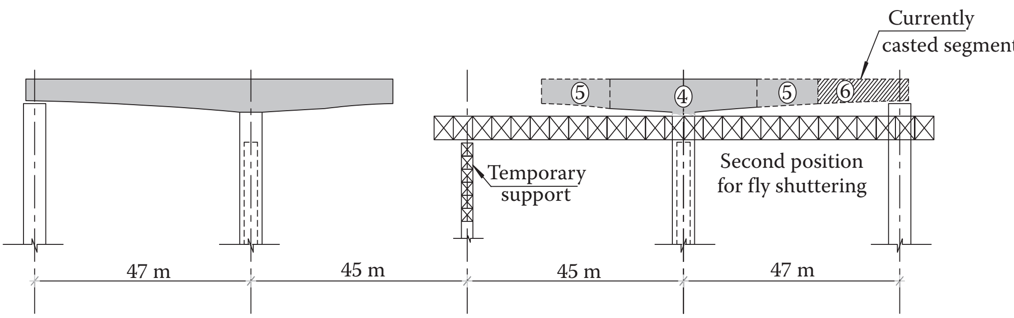
After comparing several types of superstructures for the approach bridges, continuous prestressed concrete box girder and rigid frame bridges were selected for their economic viability and structural stability.

For the span arrangement of the approach bridges, a 40-m span was noted as a standard span considering cost-effectiveness, the availability of self-launching girders, and ease of construction. Reinforced concrete piers and cast-in-situ reinforced concrete foundations were selected due to cost considerations.



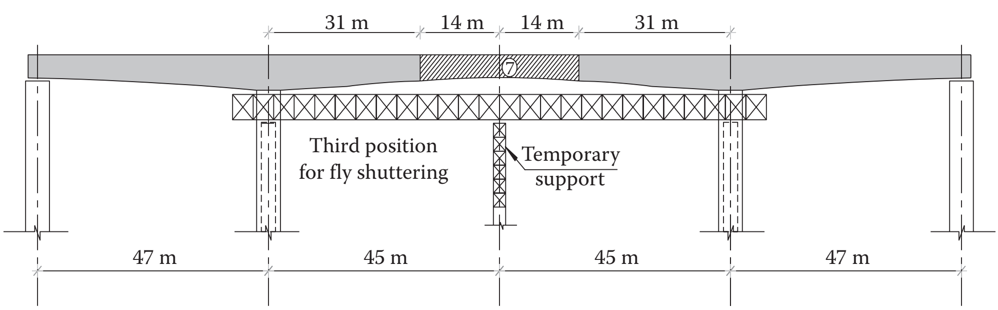
Central portion construction over the Nile (Luxor Bridge)
Stage 5

FIGURE 19.15 Casting the fifth segment.



Central portion construction over the Nile (Luxor Bridge)
Stage 6

FIGURE 19.16 Casting the terminal segment of the western part of the bridge.



Central portion construction over the Nile (Luxor Bridge)
Stage 7

FIGURE 19.17 Casting the closure segment at the center of the bridge.

The superstructure for the approach bridges consisted of two separate prestressed concrete box girders (called north and south box girders, each carrying two lanes of traffic). All the portions of the approach bridges were executed using self-launching erection girders (also known as flying shuttering, mobile, or moving scaffolding), except for some lower girders, where traditional scaffoldings were employed.

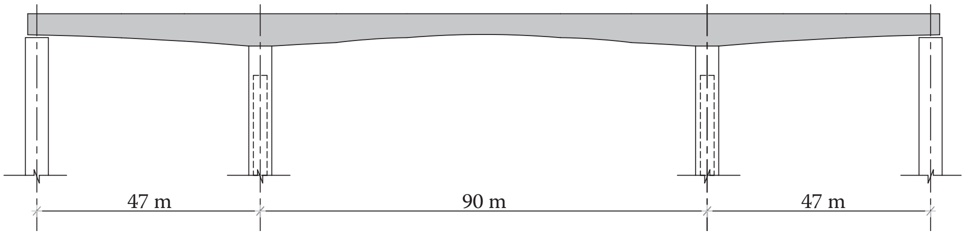


FIGURE 19.18 The central portion of the bridge after construction.

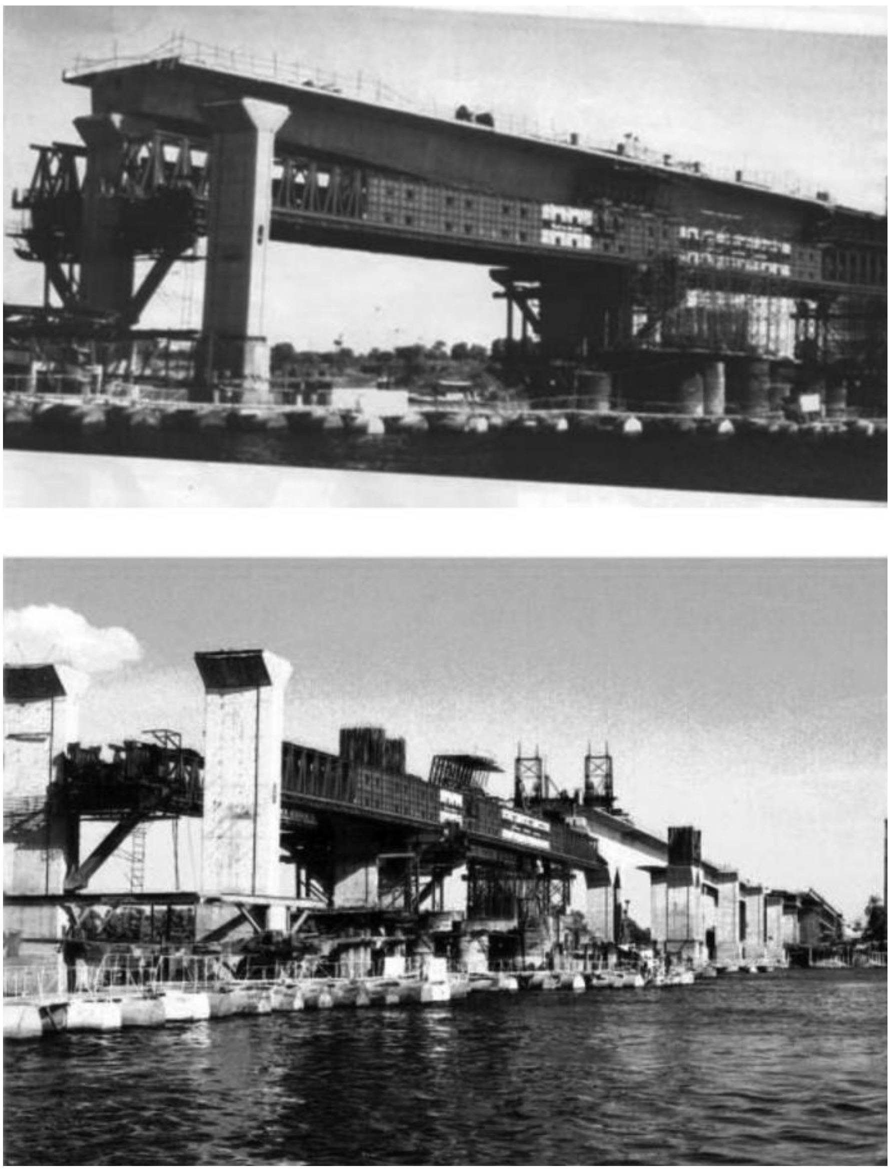


FIGURE 19.19 Movable scaffolding adopted during Luxor Bridge construction.

19.5.4.1 Eastern Suez Canal Bridge Approach Construction

19.5.4.1.1 Pier Construction

The slip form was adopted for construction of the pier columns. The slip form systems for the pier columns are manufactured, assembled, and operated by Bygging Intercontinental, Egypt. The vertical alignment is checked by optical plummets. For all pier columns, the vertical deviations were within 3 cm. The construction sequence was as follows (Figure 19.20):

Step 1: Erection of the slip form equipment and begin slipping for the first 5 m
Step 2: Erection of form for the slanted part and continue slipping for 2 m
Step 3: Removal of the form for the slanted part and erection of the inside slipping form; continue slipping until the upper slanted part
Step 4: Erection of formwork for the upper slanted part and continue slipping
Step 5: Dismantle slip form and shift to the next pier

19.5.4.1.2 Prestressed Concrete Girder Construction

The advancing shoring method was adopted for the construction of prestressed concrete box girders, because the piers are high and the box girders have constant heights and span lengths. Twin box girders are constructed simultaneously to overcome the short construction period. The advancing shoring system was designed and fabricated by RoRo Gerustbau Gmbh of Germany. The total weight is about 800 tons. The procedure of advancing shoring is described in Figure 19.21. It was assembled on the ground just beneath the first span and then lifted by jacks set on top of piers. After completing the last eastern bank span, the advancing shoring is lowered to the ground and transferred to the western bank.

The casting of the concrete is carried out in two steps: the bottom slab and webs and then the top slab. After casting the first concrete, partial prestress is applied to prevent cracks during the second pouring. Figures 19.22 and 19.23 show the eastern approach for the Suez Canal Bridge during construction. The construction sequence was as follows (Figure 19.21):

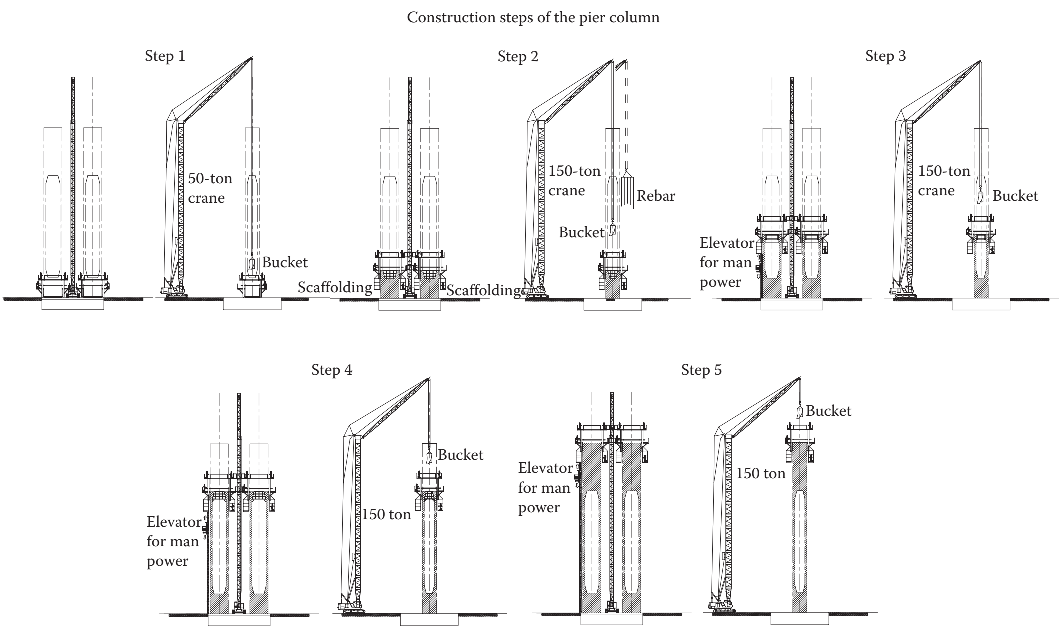


FIGURE 19.20 Steps 1, 2, 3, 4, and 5 of construction for Suez Canal Bridge.

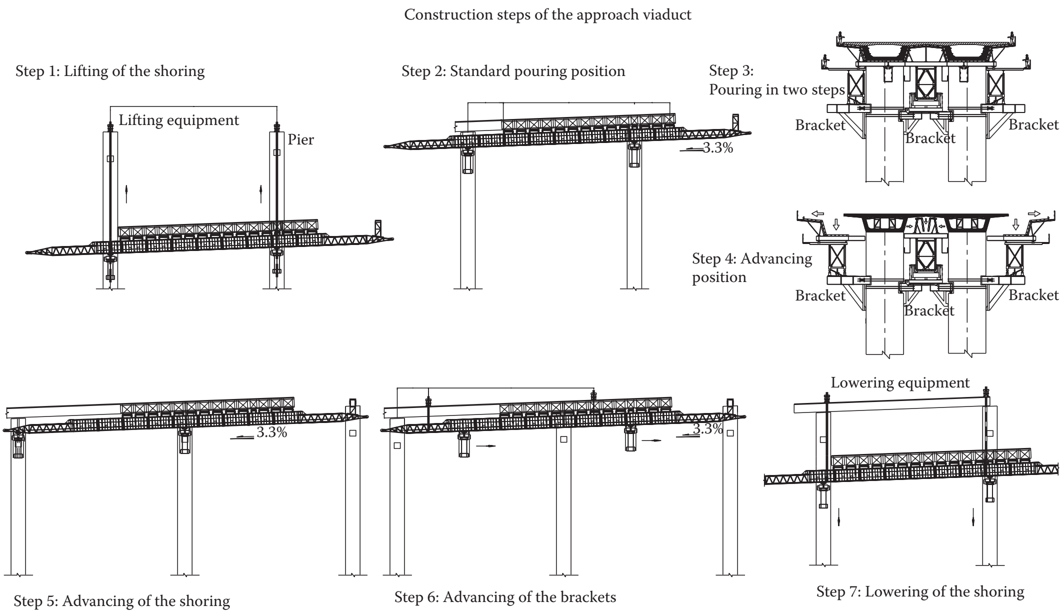


FIGURE 19.21 Steps 1, 2, 3, 4, 5, 6, and 7 of construction of eastern approach viaduct (Suez Canal Bridge).

FIGURE 19.22 Construction of the approach viaduct in the east portion (Nov. 99).

FIGURE 19.23 Mobile scaffolding for approach viaduct of the east portion.

Step 1: Lifting up the advancing shoring using the heavy lifting equipment, about 800 tons.
Step 2: Adjusting the shoring in its standard pouring position.
Step 3: Form and rebar work for the bottom slab and webs, then casting; removal of the form webs and erection of the form and rebar for the top slab and casting, then prestressing.
Steps 4 and 5: Lowering the shoring and advancing it until the center of the shoring is above the pier.
Step 6: Advancing of the brackets and suspending the shoring from the lifting. After the brackets are fixed to the pier, the shoring is moved to the next position for concreting.
Step 7: Lowering of the advancing shoring and transfer from the eastern side to the western side.

19.5.4.2 Western Suez Canal Bridge Approach Construction

The total length of the bridge is 4030 m with a cable-stayed main span 404 m long. The approaches consist of two parallel viaducts with a total width of 19.8 m. The western approach is 1.7 km long and the eastern 1.6 km. The horizontal curve is straight and the longitudinal slope is constant (3.3%). Each viaduct is made of a series of multiple span sections having a minimum span length equal to 25.45 m and a maximum span length equal to 40 m. The cross section of each viaduct consists of a box girder with a constant height of 2.3 m and variable web thicknesses from 800 to 400 mm. The bottom slab thickness varies from 400 to 230 mm. The pier height varies from 5 to 41.8 m. Figure 19.24 shows the components of the adopted stepping shuttering.

19.5.4.2.1 Launching Girder Description

A movable scaffolding system was selected by The General Nile Company for Roads and Bridges to build the six sections of the western approach, with a longitudinal beam support of the formwork system: there are two longitudinal beams for each launching girder. The length of the longitudinal beams is 103 m. Each longitudinal beam is composed of three elements: (1) main central part 50 m long, (2) front nose of 35 m with a transverse beam or front gantry, and (3) back nose of 18 m.

The main part supports all the external panels of the formwork. It is composed of five box girders of 12.5 m except for segments 1 and 2 that are separate in two parts to permit the construction of the first span. The geometrical characteristics of the built-up section are (1) constant height 2.6 m, (2) constant upper width 1.5 m, and (3) constant bottom width 1.606 m. The connection between the different segments is done by a system of prestressing bars and shear keys.

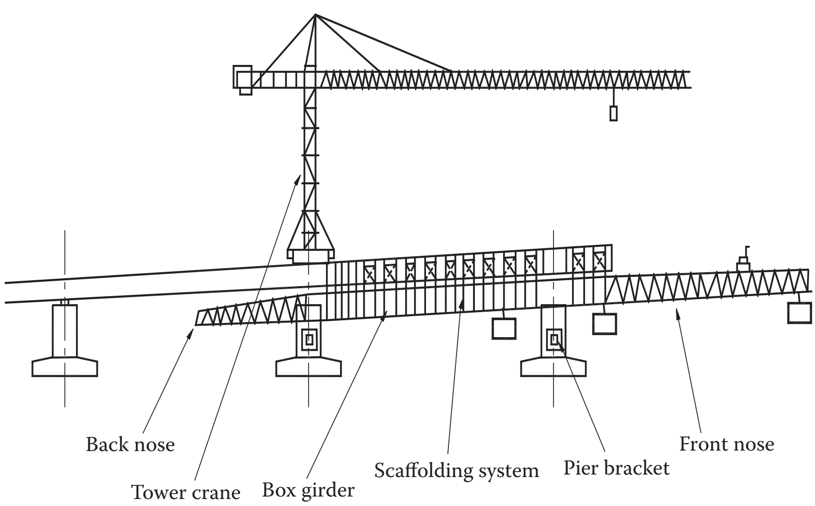


FIGURE 19.24 Shows the components of the adopted stepping shuttering.

Steel trusses are used for the front nose and the back nose. They allow a stability safety factor superior to 1.5 during the launching of the girders. The back nose has a tapered form to progressively escape the rear pier bracket during the launching. At the tip of the front nose there is a transverse beam equipped with two hydraulic jacks and two hydraulic winches permitting the lifting and the fixation of the pier bracket. This transverse beam provides a fixed distance between the two longitudinal beams (2 × 3.9 m = 7.8 m).

19.5.4.2.2 Construction Sequence

The typical construction cycle is as follows.

Step 1

- Lower the steel girder on the rollers
- Open bottom slab forms using the two operation platforms installed between the pier P − 1/P and P/P + 1 (see Figure 19.25)
- Open the external forms slightly

Step 2

- Launch the launching girder to the concreting position
- Adjust the vertical position of the dual launching girders
- Adjust the vertical position of each formwork panel
- Lower the operation platform on side P − 1/P and lift it on side P + 1/P + 2

Step 3

- Close the bottom slab forms
- Adjust the orientation of the external formwork
- During the adjustment of each panel of formwork, inject grout around the shear key of the pier brackets
- When the strength of grout is acceptable, stress at 100% the prestressing bars of the pier brackets

Step 4

- Move the tower crane, installed on the deck, to reach pier P; then with the tower crane:
  - Install the bearings
  - Install the special formwork for bearing points
  - Install the front and rear bulkhead panels

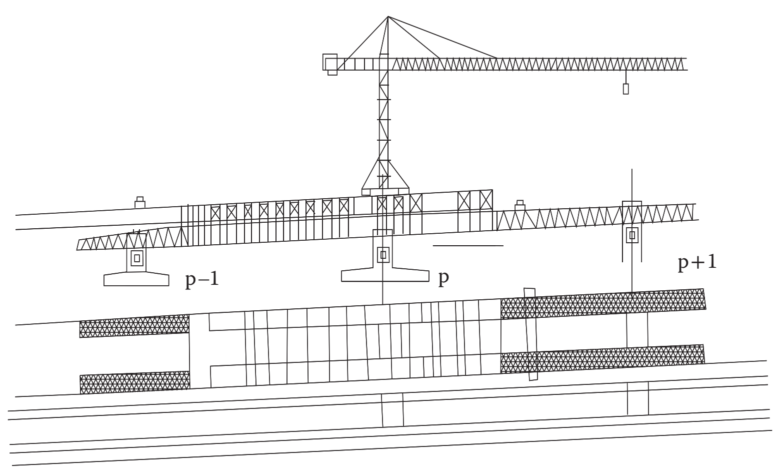


FIGURE 19.25 Suez Canal Bridge stepping form.

- Install reinforcement of the bottom slab and web
- Install longitudinal tendons

Step 5

- Install the internal web formworks with the tower crane
- Install the pier segment diaphragm formwork

Step 6

- Concrete bottom slab and webs and the pier segment diaphragm

Step 7

- Remove the internal web formwork with the tower crane

Step 8

- Install the internal upper slab formwork with the tower crane

Step 9

- Install the reinforcement of upper slab including wings with the tower crane

Step 10

- Concrete the upper slab including the joint after adjusting the vertical deformation of the launching girder (to avoid a step with the previous span)
- Apply the longitudinal prestressing when minimum specified concrete strength is reached
- Remove manually the internal upper slab formwork
- Concrete the intermediate diaphragm

Figures 19.26 and 19.27 show the movable scaffolding adopted during the construction of the western portion of Suez Canal Bridge.

FIGURE 19.26 Construction of the approach viaduct in the West Portion (Nov. 99).

FIGURE 19.27 Mobile scaffolding for approach viaduct of the West Portion.

TABLE 19.2 Comparison of the Construction Techniques of the Three Portions

	Techniques of the Three Portions		
Portion item	West side	Central	East side
Columns	Climbing shuttering	Slip-forms	Climbing shuttering
Type of form	1.5 m/day	2.5 m/day	1.5 m/day
Average rate of concreting		24 hours	
Superstructure	Two mobile scaffoldings	One mobile scaffolding	Two mobile scaffoldings
Number of scaffoldings fixation of brackets to columns	Brackets fixed to the columns mainly by friction also small shear key	Brackets supported to the columns in a recess in the columns	Brackets supported to the columns in a recess in the columns
Average rate of concreting of the superstructure	1.8 m/day	1.73 m/day	1.63 m/day

19.5.5 Comparison of Construction Techniques in the Three Portions

Every portion adopted a moving scaffolding with different features and different mounting system to the columns, as described in the following:

1. West portion mobile scaffolding: Each box girder (called north and south box girders) is casted using a separate set of mobile scaffolding. Each mobile scaffolding consists of six steel brackets supporting two steel trusses, which in turn support the formwork of the box girder. Note that only four brackets are needed to support the mobile scaffolding. The other two brackets are used to speed up construction. Fixation (mounting) of the steel brackets to the columns is mainly done through friction between the surfaces of the steel plates of the brackets and the concrete columns. The friction is provided by tensioning six bars at each column (small diameter horizontal wholes are made during casting of the columns, to allow insertion of the bars, and tensioned at the time of mounting the brackets to the columns). Steel shear keys are also provided for alignment and to help in load transfer through bearing on the concrete columns at the lower sides of the shear keys.
2. East portion mobile scaffolding: Each box girder (north and south) is cast using a separate set of mobile scaffolding. The load of the mobile scaffolding is transferred to the concrete columns by steel brackets, supported in a large recess in the columns, in addition to six tension bars (three on each side of the column, at the outer face, i.e., outside the column cross section).
3. Central portion mobile scaffolding: Both box girders (north and south) are cast simultaneously using one set of mobile scaffolding. The scaffolding has the same features and main principal elements as the east mobile scaffolding, except that it is prepared and furnished to concrete the two boxes of the superstructure in the same cycle, that is, the total width of the superstructure is cast in one cycle.

Table 19.2 shows comparisons of the adopted construction techniques for the east, west, and central portions of the Suez Canal Bridge.

19.6 Balanced Cantilever Construction (Cast-In-Situ Technique)

19.6.1 Cantilever Construction by In-Situ Concreting Using Two Form Travelers (Concreting Carriages)

In this method, the superstructure is constructed in segments using two form travelers, one at each side of the bridge superstructure. This method is used mainly when there are obstacles on the ground. It has been used successfully in many bridges over the Nile in Egypt. It can be used regardless of the ground condition. Cranes are required to erect and dismantle from travelers and during operation. The bridge is executed in segments, as illustrated earlier. The length of each segment usually ranges between

3 and 5 m. The system is integral, in the sense that it requires limited help from other equipment except for handling construction material, erection, and dismantling. This method can be classified as machine intensive; thus, it is relatively expensive and requires high initial investment. The travelers can be used several times for similar jobs after modification. The precision required in this method mandates using highly skilled labor. It is obvious that the construction stages have a direct bearing on design.

Construction starts from bridge piers to the cantilever ends as follows:

1. The first section of the bridge superstructure that is constructed is called the stump, as illustrated in Figure 19.28. The length of the stump usually ranges between 10 and 15 m and there are a number of methods used in constructing it, either using stationary falsework and formwork or using Bailey panels to support the falsework and formwork.

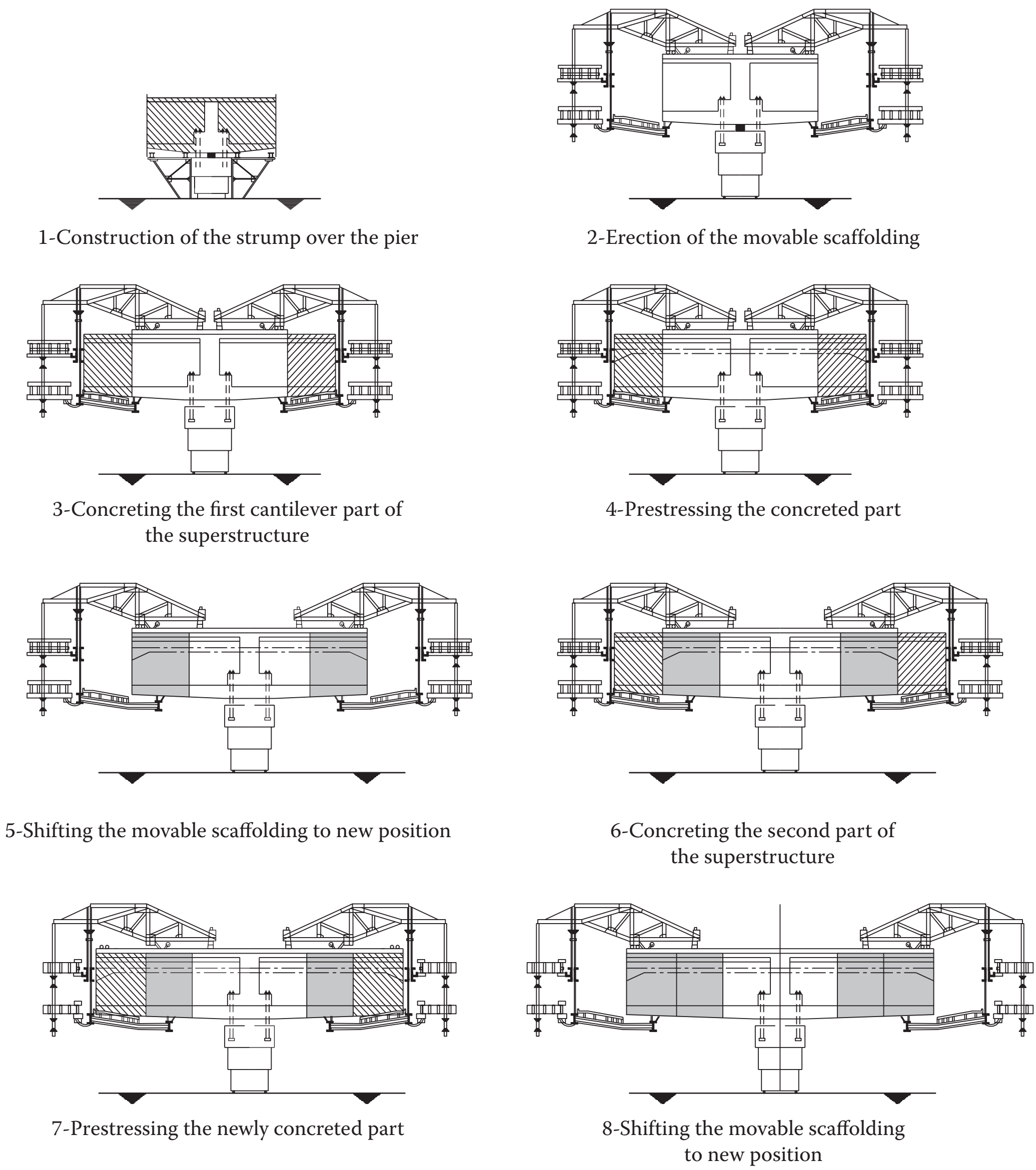


FIGURE 19.28 Construction stages for balanced cantilever construction.

2. The form travelers are erected on both sides of the stump and moved progressively. Steel reinforcement and prestressing ducts are installed for each segment. The segment is concreted and after concrete gains the required strength prestressing cables are stressed and the form travelers move to stage 3. As construction proceeds, the continuity cables are inserted progressively and stressed across segments. The work proceeds symmetrically from both ends of the cantilever to ensure balance as illustrated in Figure 19.28.
3. One of the two form travelers is dismantled and the other one advances to support the central gap at midspan. Steel reinforcement and prestressing ducts for the bottom slab are installed and then concreting takes place. The prestressing cables in the bottom slab should be stressed to cater to continuity stresses and the remaining form traveler is removed from the bridge. Figure 19.29 illustrates all the stages of the bridge construction up to the last stage when casting the closure segment.

There are two important issues that should be highlighted:

1. Careful consideration for camber calculations is required to ensure that the two sides of the superstructure and central gap match. The contractor must also observe any differences during construction and report them to the designer so that calculations can be adjusted.

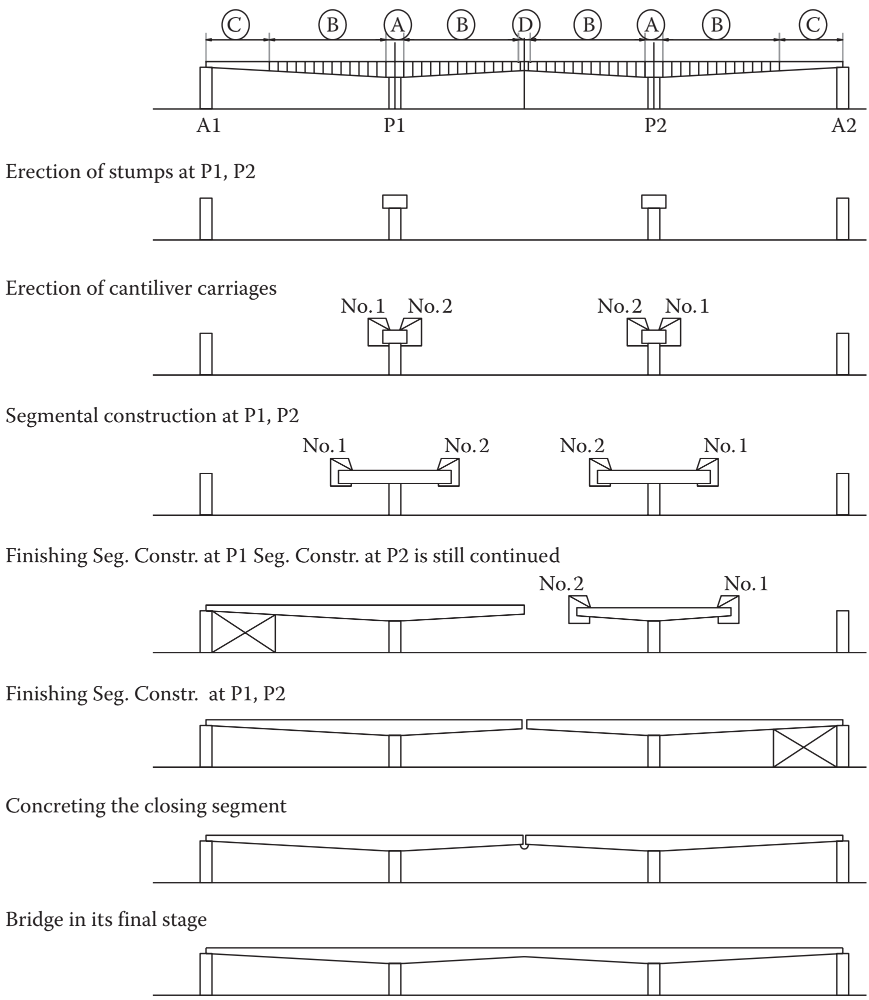


FIGURE 19.29 Bridge construction using balanced cantilever method.

2. The superstructure must be fixed to bridge piers during construction. If not, temporary fixation is required. This connection can be removed after finishing the central gap.

19.6.2 Cantilever Construction by In-Situ Concreting Using One Form Traveler and Stationary System

It may be possible in some instances to use only one traveler and stationary formwork if the ground conditions are favorable to reduce cost or to overcome limited resources. The main components in this method are one form traveler and stationary system. Concrete blocks are normally used to balance the weight of the form traveler on the other side. The construction sequence is as follows:

- The stump is constructed using one of the previously stated methods described in Section 19.6.1.
- After constructing the stump, the form traveler is erected at the same time with stationary falsework on the other side.
- To counterbalance the effect of the loads resulting from the form traveler's weight, a number of concrete blocks are placed on the previously finished section of the superstructure, as illustrated in Figure 19.30. The blocks are mounted on steel I-beams on the top slab. The number and arrangement of the blocks should be determined by the designer. As the form traveler moves forward to the next position, the concrete blocks are transported to the next section on the other side of the cantilever.

The Construction of concrete bridge with cast-in-situ cantilever method is popular in Egypt, especially for the bridge over the River Nile.

For example the Arab contractor (Osman Ahmed Osman & Co.) completed more than 16 bridge using this construction method in Egypt.

Figures 19.31 and 19.32 show examples for some of construction stages of Rod El-Farag Bridge in Cairo. (Figures are based on Arab Contractors website: http://www.arabcont.com/english/services/BridgesSystems.aspx).

A key issue in the construction of balanced cantilever bridges is preserving the balance between the two cantilevers at both sides of the bridge. Figure 19.33 shows example of fixation of deck to pier in cantilever bridges.

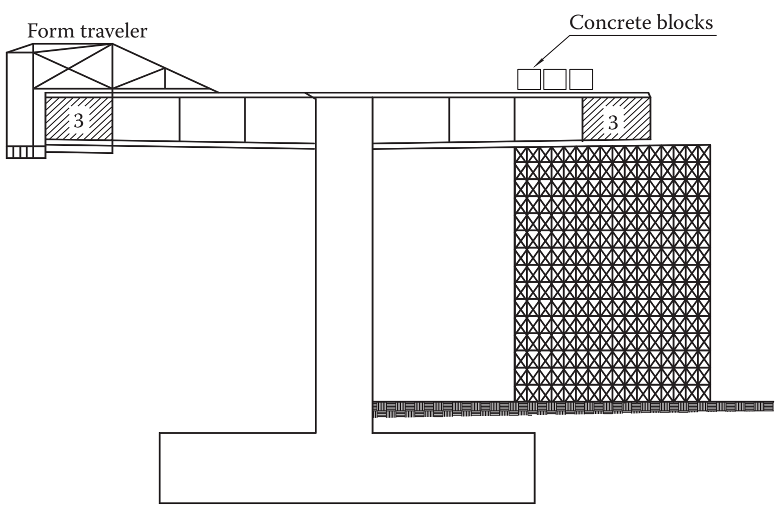


FIGURE 19.30 Using one form traveler and a stationary system.

FIGURE 19.31 Moving travelers on the stump (Rod El-Farag Bridge).

FIGURE 19.32 Casting of segment (Rod El-Farag Bridge).

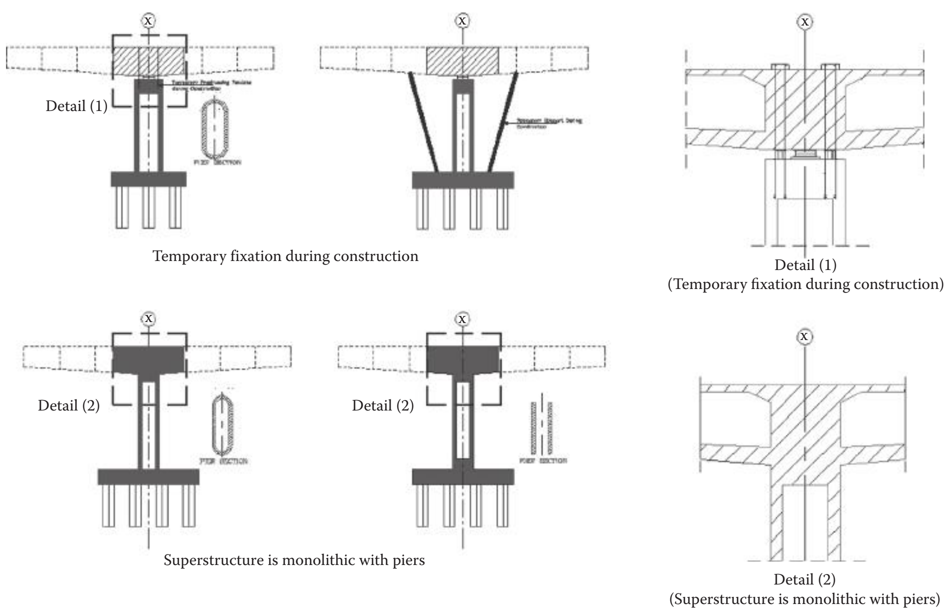


FIGURE 19.33 Example of fixation at deck to pier in cantilever bridges.

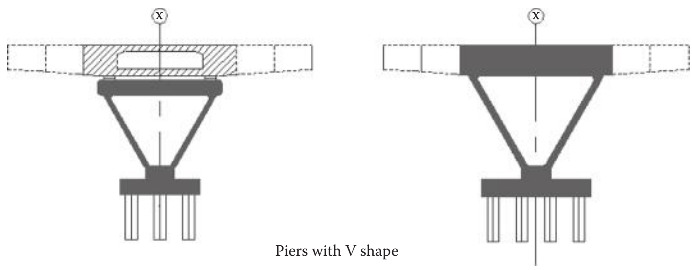


FIGURE 19.33 (*Contiuned*) Example of fixation at deck to pier in cantilever bridges.

19.7 Horizontal Incremental Launching (Deck Pushing System)

The key element here is preserving the balance between the two cantilevers at both sides of the bridge. Figures 19.31 and 19.32 show some of the construction stages of Rod El-Farag.

19.7.1 General

This system works by pushing previously casted concrete elements from the casting area to the required span. The main components are temporary columns to support the formwork at the casting area; a steel nose is normally used to minimize the cantilever length of the segments during pushing, accompanied by a system of jacks. Other accessories are also used such as Teflon sheets and grease. The method is used for box cross sections. It can accommodate limited constant horizontal curvature of the deck. It works regardless of the ground, so it can be used for any height superstructure except for the casting area, where the height should enable casting the segments. The segment length usually ranges between 15 and 30 m. The bridge is pushed in the direction of down grade (if available) to reduce resistance to pushing. This method is used for prestressed concrete.

This method has been used in downtown areas successfully as it causes minimum interference with the surrounding environment. It can be used to cross most obstacle types. Ground conditions have limited impact as the system is totally supported over bridge piers normally. Although this method can be described as machine-intensive, it is not generally expensive. This method requires skilled labor and experienced staff and the design is highly affected by the construction stages. The construction sequence is as follows:

- A casting area is prepared at one end of the bridge. Two casting areas may be required, one at each end of the bridge, depending on the bridge length. After concreting the segments, the formwork is stripped and a lightweight steel nose is fixed to the segment front to limit the moments due to the cantilevering effect, as illustrated in Figure 19.34. Bridge bearings are provided with temporary sliding bearings made up of steel or concrete with a stainless steel surface and side guiding plates to keep the superstructure to the correct alignment. The bearings are coated with Teflon on the sliding surface to facilitate sliding with minimal friction.
- The formwork in the transverse direction is supported on four temporary columns and two temporary beams. Each segment is usually concreted in two stages, the bottom slab and webs and then the top slab. The temporary beams are equipped with hardwood at the top to facilitate the pushing process, as shown in Figure 19.35.
- After pushing the first segment to the required position, the second one is concreted over temporary columns, the segments are prestressed, and the same process is repeated, as illustrated in Figure 19.36.
- After finishing the prestressing, the temporary bearings should be replaced with permanent ones. However, the modern practice is to use permanent bearings during construction as well.

Figure 19.37 shows some of the construction stages using the deck pushing system technique for Zamalek Bridge in Cairo.

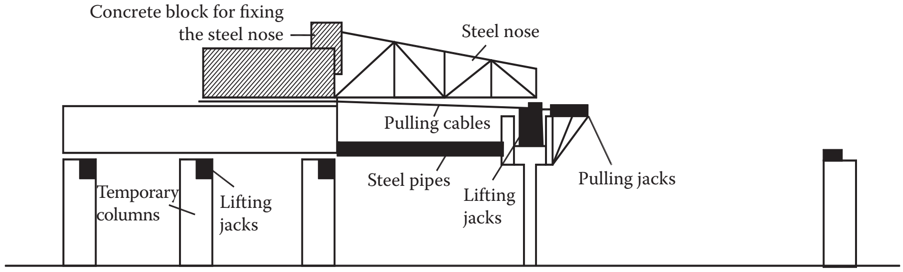


FIGURE 19.34 Preparation of pulling segment.

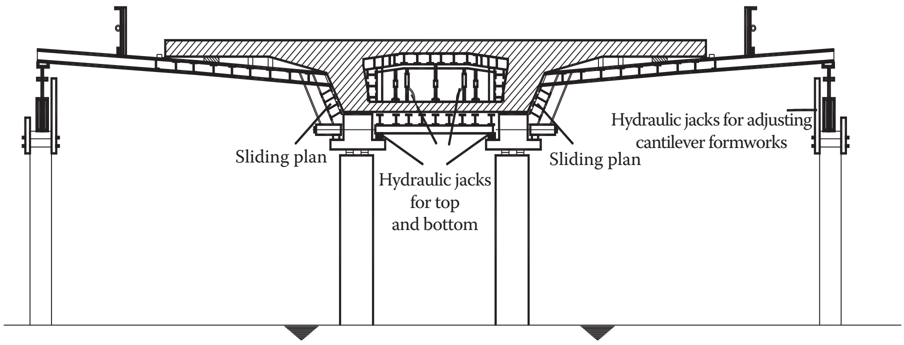


FIGURE 19.35 Transverse section in fabrication area before lowering formwork.

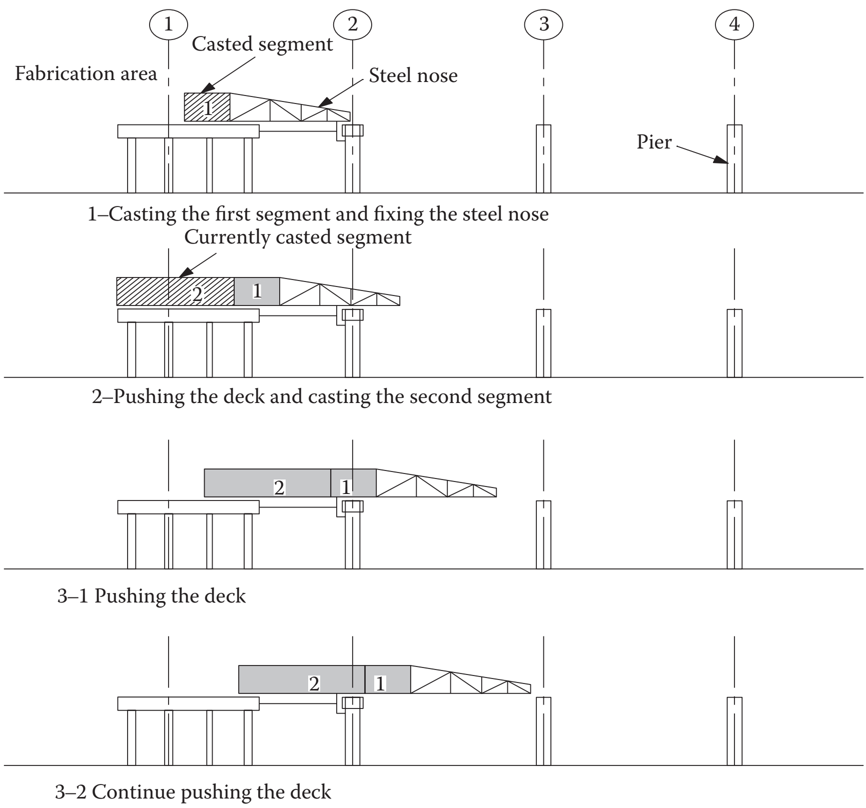


FIGURE 19.36 Successive steps of construction using deck pushing method.

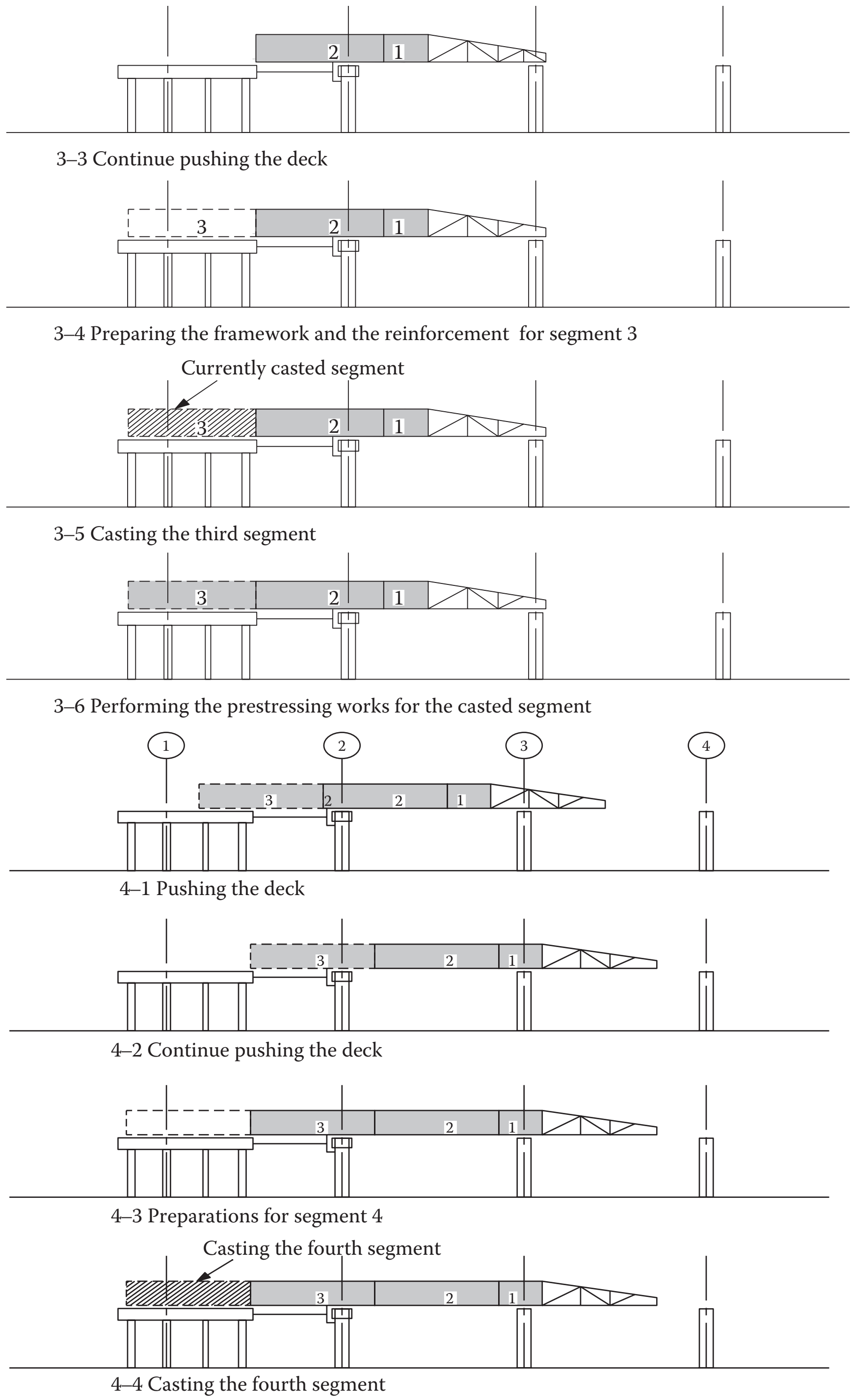


FIGURE 19.36 (*Continued*) Successive steps of construction using deck pushing method.

FIGURE 19.37 Deck pushing system in Zamalek Bridge, Cairo, Egypt.

19.8 Launching Truss Method (Precast Elements)

19.8.1 General

The system normally consists of two trusses that are supported over the bridge pier heads. The trusses are supported over steel chairs and are equipped with moving hoists that travel freely over them. The precast beams are transported from the casting yard to the required span using two transport trolleys. The main components of the casting yard are generally a number of molds and a gantry crane. The main components of the launching system are illustrated in Figure 19.38.

This system is used for erecting prestressed precast beams of beam and slab bridges. It is used for the erection of straight-line bridges, although very limited horizontal curvatures may be accommodated. It works regardless of the ground, so ground conditions have a limited effect on the construction process. It is normally used for spans ranging between 30 and 60 m. It can be used to cross any type of obstacle, although it has been used most successfully in downtown areas and to cross small waterways. The method requires little interference with the public, so it can be used efficiently in crowded areas or where there is traffic.

19.8.2 Ghamra Bridge Construction In Cairo Using the Launching Truss Technique

The construction process of the bridge passed through repeated cycles. Each consisted of the following steps:

1. Casting of the girders
2. Partial stressing for transportation purposes
3. Transport of girders to their spans
4. Transport of girders to the launching truss
5. Lowering of girders to piers and positioning them into final position on bearings
6. Casting of the deck slab and cross girders
7. Stressing of remaining cables
8. Advancing of launching truss to the next span

The main system components were

- Manufacturing area
- Gantry cranes to lift girders
- Transport carriages
- Launching truss equipped with two hoists

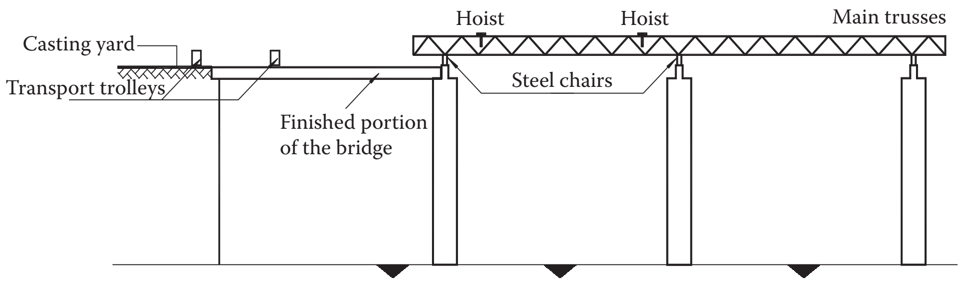


FIGURE 19.38 Main components of launching truss system.

19.8.2.1 Launching Truss Description

The launching truss is designed to carry girders up to 150 tons in weight and 45 m length and to work on vertical slopes up to 6% and also to bear wind speed up to 72 km/h. It makes three kinds of motion: a longitudinal motion that must be on a straight line, a transversal motion that must be on a horizontal level, and a rotational motion that must be done while the launching truss is resting on two piers and the motion is worked through the use of Teflon sheets. In the case of vertical curves in the bridge and differences in the elevation of the supporting piers, the support must be lifted by the use of hydraulic jacks. In the case of horizontal curves, there is a superelevation in the pier, and the horizontal level can be made by using chairs.

The launching truss is composed of the following elements:

- Main truss: The main space is truss 106 m long, consists of two vertical in-plane trusses 3.65 m high and spaced by 5.7 m and connected at the two ends by two portals. Each truss is made of eight elements, each 12 m long, and two curved terminal sections of 5 m each.
- Hoist bogies: Each hoist bogie is formed by a supporting frame running on four wheels on each side. On the upper profile of trusses there is an electric motor mounted on the first hoist, which is used to move either the launching truss or the two hoists. There exists also another electric motor to give the relative motion of the two hoists. For anchoring the hoist, there is a double system of ropes.
- Rolling systems and rails: The roller unit moves on the rails, which are supported on piers to allow the transversal motion. Under the rails, chairs are found, which give the horizontal level. Three roller units allow the three kinds of motion.

19.8.2.2 Construction Sequence

Step 1: The launching truss on three supports is anchored to roller groups located on pier 2 (P_2) through carriage C_2, and hoists A_1 and A_2 are located as in Figure 19.39. The transport vehicle (MAFI) supporting the girder stops next to hoist A_2, which hooks and lifts the girder up.

Step 2: Hoist A_2 moves forward so as to allow hoist A_1 to engage the rear part of the girder, as shown in Figure 19.40.

Step 3: Hoists A_1 and A_2 move forward holding the girder and stop in the final position between the two piers, as shown in Figure 19.41.

Step 4: Hoists are anchored to roller groups R_2 and R_3 with the anchor devices X_1 and X_2. Carriage C_2 is released and now the launching truss is free to move forward, as illustrated in Figure 19.42.

Step 5: The launching truss is stopped in the new position, symmetrically located between piers. The launching truss is fixed by anchoring carriage C_1 to roller groups R_2, as shown in Figure 19.43.

Step 6: Finally, anchors X_1 and X_2 are released, as shown in Figure 19.44. All transversal and rotational movements of the launching truss are now possible. Other girders are first erected as they ask for larger space.

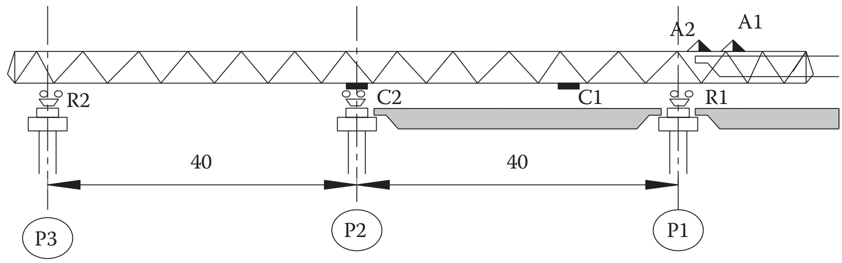


FIGURE 19.39 Step 1 of construction stages.

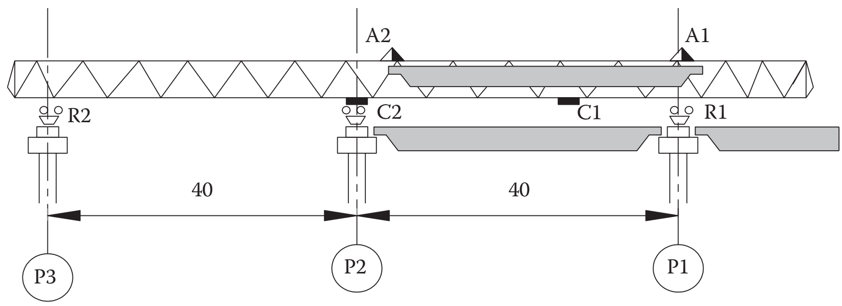


FIGURE 19.40 Step 2 of construction stages.

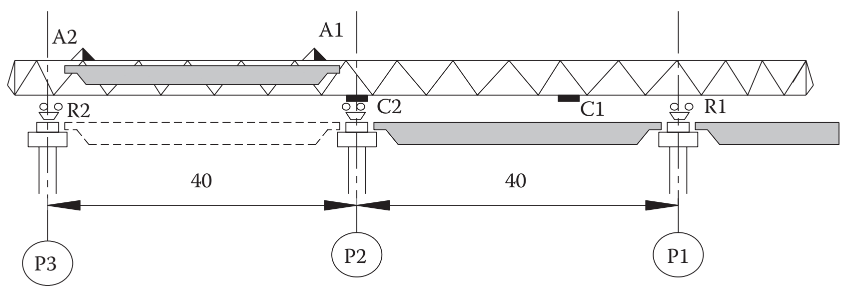


FIGURE 19.41 Step 3 of construction stages.

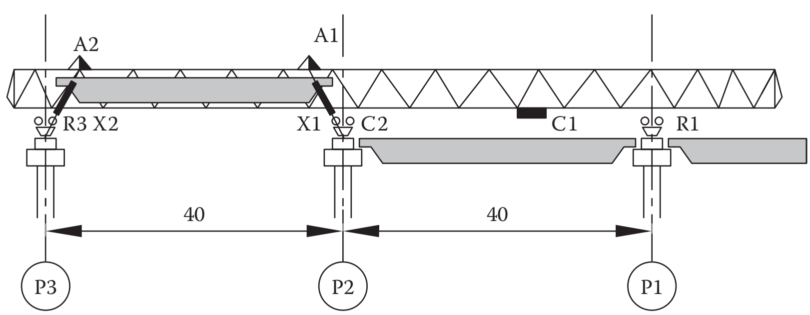


FIGURE 19.42 Step 4 of construction stages.

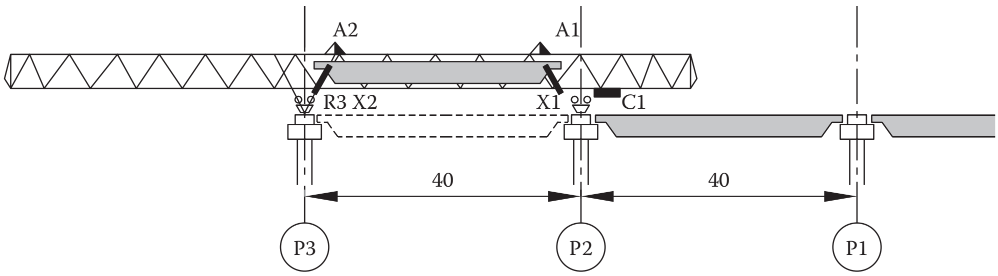


FIGURE 19.43 Step 5 of construction stages.

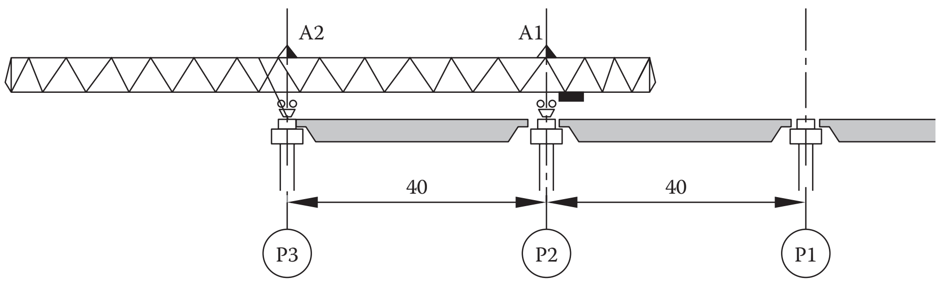


FIGURE 19.44 Step 6 of construction stages.

19.9 Erecting Bridge Elements Using Cranes or Heavy Lifting

19.9.1 General

This system uses either cranes or heavy lifting to erect the beams. One or two cranes may be used depending on the beam weight, length, and erection height. The main components are hydraulic jacks, high-tension steel bars, and steel beams. The number and arrangement of jacks and bars is dictated by the beam weight. This method can be used for steel beams comprising orthotropic decks as well as for concrete beams whether prestressed or not. The beams can be produced curved and then lifted into position to accommodate the required curvature in the deck. The workable height from the ground depends on the available crane capacity. On the other hand, heavy lifting is not affected by height, although proper consideration of wind should be taken.

This method may suit a crowded area if special arrangements for the working hours are considered. It may be used in the presence of obstacles if the available cranes are suitable. Heavy lifting can be used in the case of obstacles, provided beams can be placed under the span. It requires skilled laborers due to the delicacy of the process. Sufficient area for crane maneuvering is required and the ground should be strong enough to sustain crane loads. There are virtually no considerable loads created on the structure that would affect design except in the case of heavy lifting. The process is straightforward but requires skilled laborers. The time required is considerably less than other methods, especially for a limited number of spans. The construction sequence is as follows:

1. The lifting jacks are erected above the superstructure and the lifting bars are extended to the ground level and attached to the beams, as illustrated in Figure 19.45.
2. The jacks lift the beams in stages defined by their maximum stroke. The jacks are allowed to release after reaching maximum stroke to start another stage.
3. When using cranes, the procedures are simpler. The method is simple and is comprised of lifting the beams using one or two cranes depending on the beam loads, span lengths, and required height.

19.9.2 Application in El-Moneeb Bridge

This construction method of lifting bridge beams using cranes was used during construction of El-Moneeb Bridge in Cairo. It is considered the largest bridge crossing the Nile in length, width, and width of navigation spans. It connects Cairo in the east and Giza in the west. It is one of the most important axes to the ring road around greater Cairo. This bridge connects Alexandria Desert Road, Fayoum, 6th of October City, the east and west banks of the Nile, South Giza, and Upper Egypt. The bridge has six inlet and six outlet ramps. The length of the bridge is 2 km and its width is 42 m. Figures 19.46 through 19.48 illustrate the erection of steel bay beams using lifting cranes for El-Moneeb Bridge.

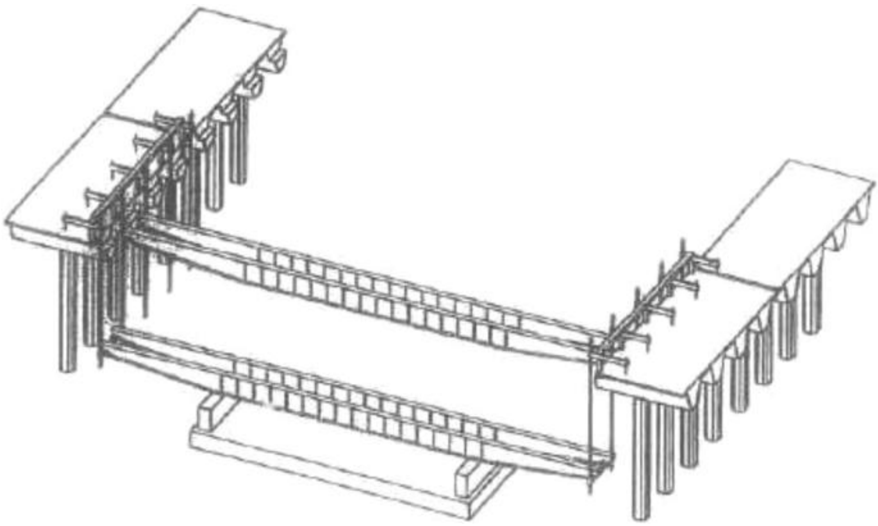


FIGURE 19.45 Heavy lifting of beams.

FIGURE 19.46 Steel beams to be supported by the brackets (El-Moneeb Bridge).

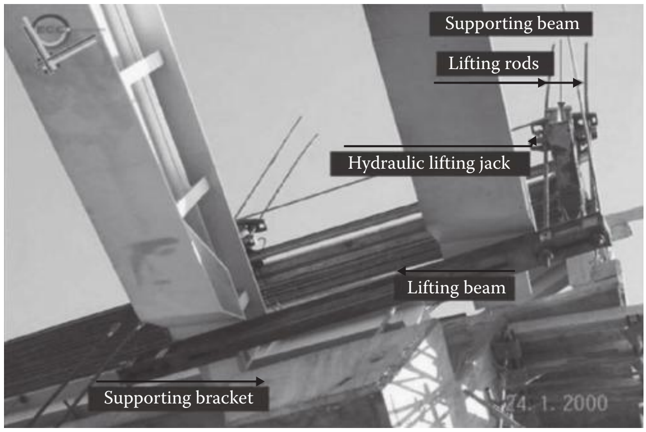


FIGURE 19.47 Steel beams after erection (El-Moneeb Bridge).

FIGURE 19.48 First group of steel beams during lifting (El-Moneeb Bridge).

19.10 Construction of Precast Concrete Segmental Bridges

This section presents several examples in the construction of precast concrete segmental bridges (PCSB). The drawings in this section are based on VSL Brochure 2013. Excellent information on progress and previous experience in PCSB is available in Podolny and Muller (1982), ISTED (1987), and ASBI (2008).

1. Precast segmental construction balanced cantilever erection with launching gantry
2. Precast segmental construction span-by-span erection with launching gantry
3. Precast segmental construction balanced cantilever erection using lifting frames
4. Precast segmental construction balanced cantilever erection using cranes
5. Precast segmental construction span by span erection on falsework

19.10.1 Balanced Cantilever Erection with Launching Gantry

The balanced cantilever erection with launching gantry offers savings in lifting equipment. Bridge segments are delivered along completed deck, and most works proceed above the terrain as well; therefore, no disruption is caused to existing traffic. All temporary loads are introduced directly into piers. Refer to Figure 19.49.

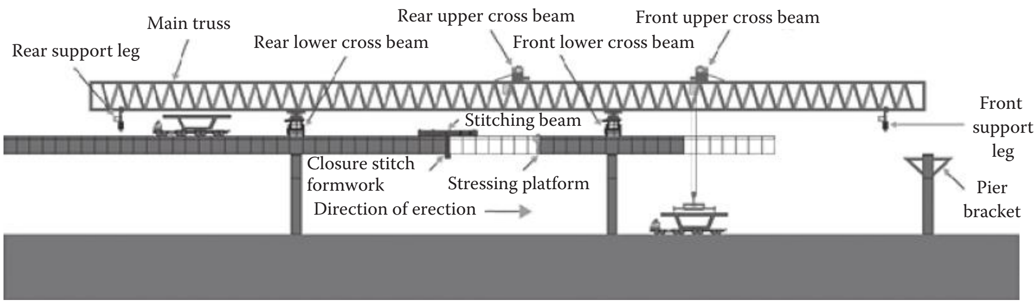


FIGURE 19.49 Example of balanced cantilever erection with launching gantry. (From VSL. *Bridge Construction Partner*, VSL International, Köniz, Switzerland, 2013.)

19.10.2 Span-by-Span Erection with Launching Gantry

Segments are installed en bloc along the whole length of span, then prestress is introduced into the structure through external tendons, which results in faster construction compared to the balanced cantilever erection with launching gantry, with less workers required. With overhead gantry, VSL regularly achieves 1 span per 2.5 days; with underslung gantry it is 4 days for 1 span. Refer to Figure 19.50.

19.10.3 Balanced Cantilever Erection with Lifting Frames

The delivery of segments to the place of installation basically influences the selection of cantilever erection method. For projects with considerable delivery constraints, VSL has successfully started to use the balanced cantilever erection with lifting frames. Refer to Figure 19.51.

Following assembly, installation and commissioning by suitable cranes, VSL lifting frames can operate independently. They are equipped with work platforms providing unlimited access to all necessary erection fronts. Furthermore, they incorporate two primary hydraulic systems providing both lifting of segments and moving of the frame itself during construction. When high lifting speeds are required, generally greater than 20 m/h, tandem lifting units or winch systems are utilized on lifting frames. VSL technical centers typically work with engineers to incorporate limiting aspects given by the lifting system into the permanent structure design, to ensure suitable load introduction into the permanent structure and also provide the structure analysis, geometry control, and pre-camber calculation in all phases of construction. VSL regularly achieves erection cycle of two pairs of segments per shift.

19.10.4 Balanced Cantilever Erection with Cranes

When site and ground conditions are suitable, this method has proved highly effective and can facilitate considerable rates of erection. Other advantages include a relatively easy availability of cranes at local market, their possible subsequent utilization on site, minimum requirements for temporary structures, and optimization of labor forces.

VSL project teams provide detailed craneage studies helping to optimize crane usage, while minimizing disruption to the surrounding environment. VSL technical centers work with production teams to develop additional temporary works, in particular pier segment supports and work platforms. VSL is also capable of offering the structure analysis in all phases of construction, including detailed pre-camber calculation. With careful project design and management, VSL has achieved excellent results of construction. Typical erection cycle is six segments per day. Refer to Figure 19.52.

19.10.5 Span-by-Span Erection on Falsework

If the height and weight of structure permits, segments can be erected on falsework structure. Hydraulic systems are used for temporary support of segments and their alignment. Typically, cranes are used for

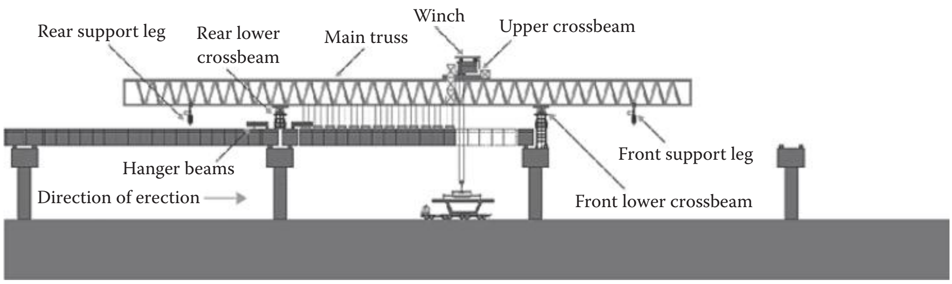


FIGURE 19.50 Example of span-by-span erection with launching gantry. (From VSL. *Bridge Construction Partner*, VSL International, Köniz, Switzerland, 2013.)

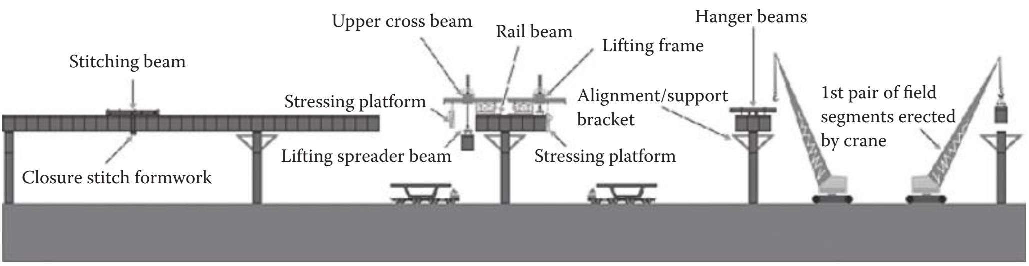


FIGURE 19.51 Example of balanced cantilever erection with lifting frames. (From VSL. *Bridge Construction Partner*, VSL International, Köniz, Switzerland, 2013.)

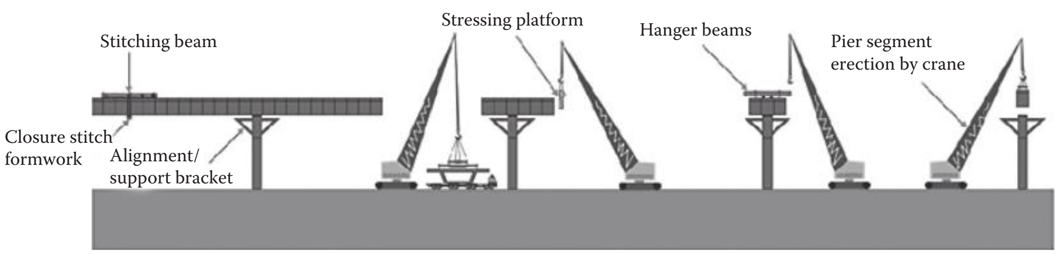


FIGURE 19.52 Example of balanced cantilever erection with cranes. (From VSL. *Bridge Construction Partner*, VSL International, Köniz, Switzerland, 2013.)

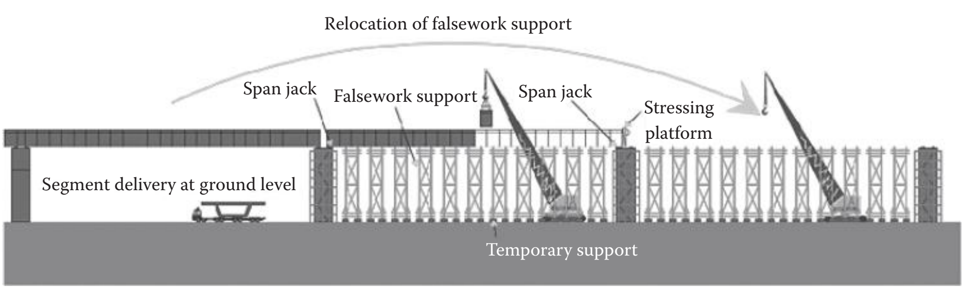


FIGURE 19.53 Example of span-by-span erection on falsework. (From VSL. *Bridge Construction Partner*, VSL International, Köniz, Switzerland, 2013.)

loading segments onto the falsework support. Nevertheless, if access to the place of loading is limited, VSL has developed alternative solutions for the handling and loading of segments. VSL uses modular support systems enabling fast relocation of support structure to another span of the structure being constructed. Typical erection rate achieved by VSL is one span per 3 days. Refer to Figure 19.53.

19.11 Bridge Rotation

Structural rotation is a construction method that allows for structures to be put in place without disrupting traffic (road or river). The structure is built parallel to the road or waterway to be crossed and then moved into its final position by rotation. During the operation, the structure rests on special bearings.

An example on bridge construction by rotation is provided in *Soils & Structures*, The Freyssinet Group Magazine, No. 225 First half 2007, Vélizy, France, pp. 28 & 29.

> One pivot bearing that acts as the point of rotation and bearings sliding on stringers, often by means of neoprene pads. The technique is well known at Freyssinet, as it was notably used in 1991 for the Pont des Martyrs in Grenoble, France, and in 2001 for the Cernavoda Bridge over the canal linking the Danube to the Black Sea in Romania. Refer to Figure 19.54.

19.12 Full-Span Precast Method (Heavy Lifting)

The full-span precast method of erection is suited for specific structures comprising multiple spans of similar lengths and minimum curvature. Interesting examples of this construction method are illustrated in Figure 19.55 (VSL 2013).

FIGURE 19.54 Example of bridge rotation (Freyssinet).

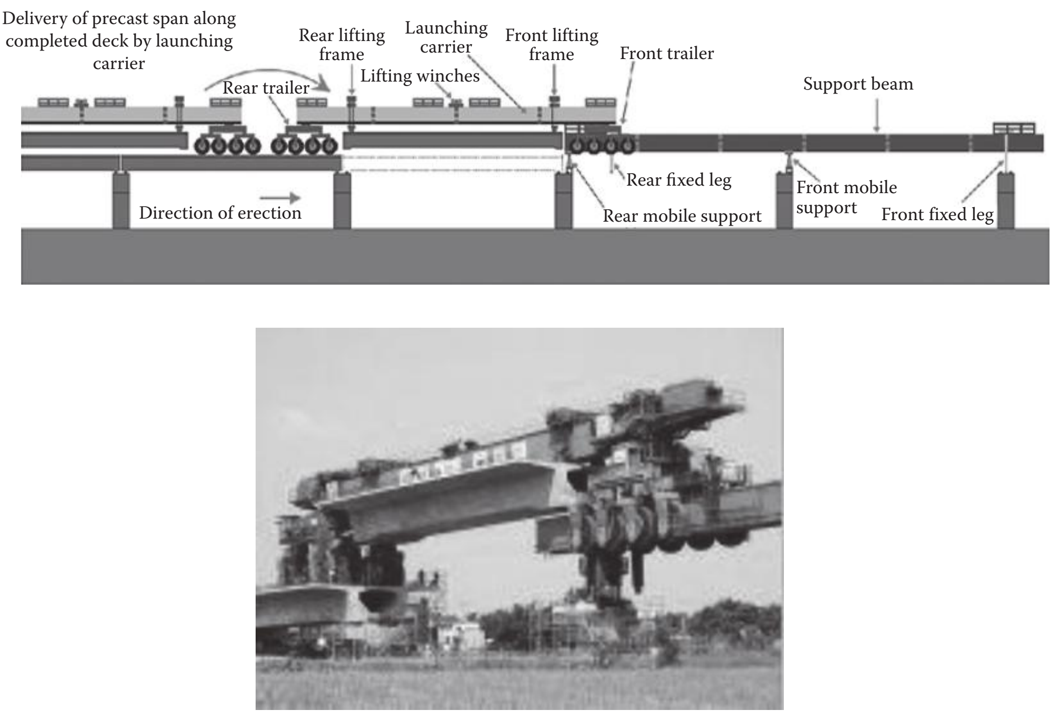


FIGURE 19.55 Example of full-span precast method. (From VSL. *Bridge Construction Partner*, VSL International, Köniz, Switzerland, 2013.)

Full-span precast elements can be made under factory conditions that improve safety, precision, and efficiency of works. Besides work platforms at pier heads, only very little temporary structures are required on-site. The delivery of manufactured spans along the already made part of the bridge structure in no way disrupts the existing road traffic, and no terrain improvement works are necessary that would otherwise be required if spans are transported at ground level.

The achieved rate of erection for the preceding mentioned project was nearly two spans per day on average.

19.13 Example of Steel Bridge Construction

The methods of construction for steel structures as shown in Figure 19.56 can be classified as follows (Nakai and Yoo 1988):

1. Staging erection method, see Cases 1 and 2
2. Erection method using erection girder or truss, see Cases 3 and 5
3. Cantilever erection method, see Case 4
4. Large assembled erection method, see Case 3

19.14 Bridge Foundations in Water

Methods used to construct underwater foundations include caissons and piled foundations with a pile cap. To construct pile caps under water, the use of cofferdams is essential to exclude water from the working area. Steel sheet piling is widely used for cofferdams because of its structural strength, water

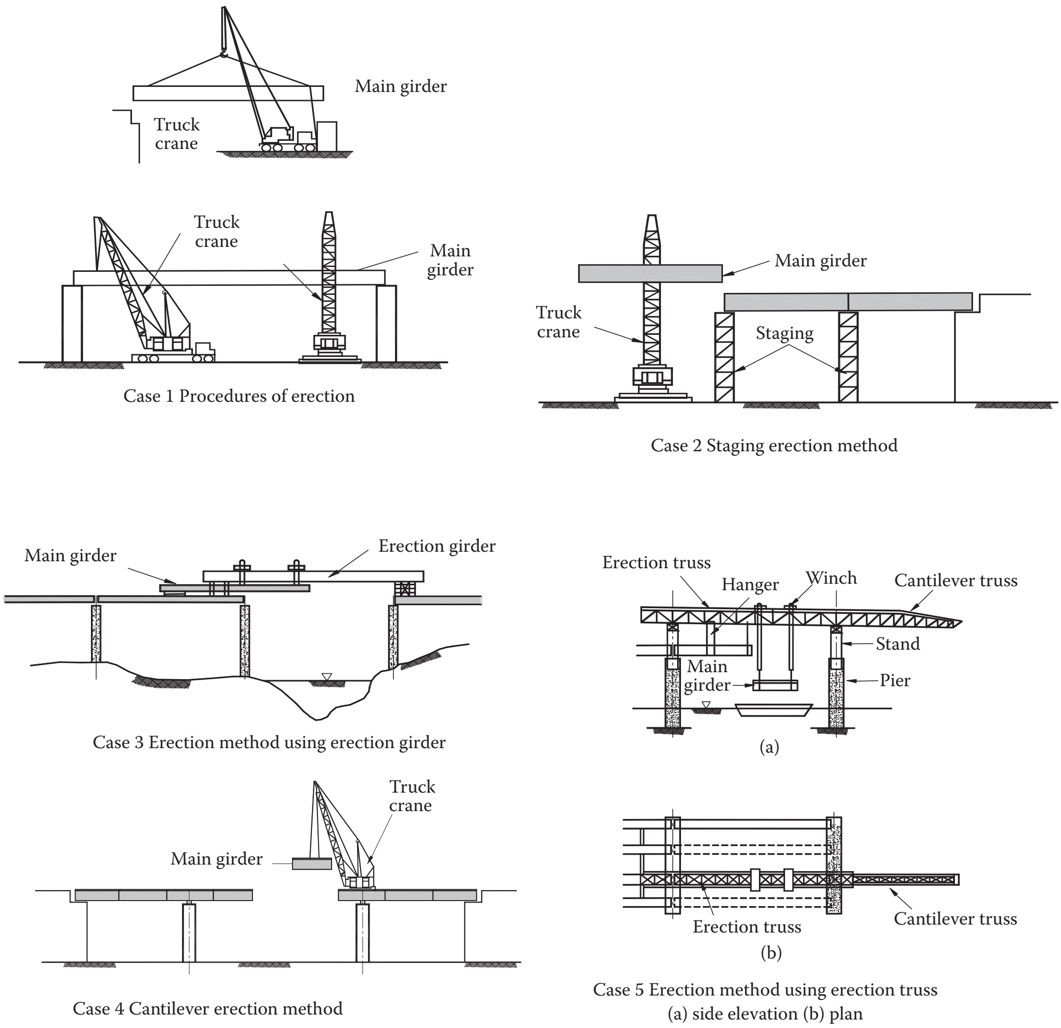


FIGURE 19.56 Example of methods of construction for steel structures.

tightness given by its interlocking sections, and ability to be driven to deep penetration in most types of ground. The main drawbacks associated with using cofferdams are that (1) the cost of cofferdams or other temporary formwork might greatly exceed the cost of the permanent foundation structures and (2) security against blows or piping can be obtained only at a high cost.

19.14.1 Pile Foundations Using Sheet Piles

19.14.1.1 General

Sheet piling is a form of driven piling using thin interlocking sheets of steel to obtain a continuous barrier in the ground. The main application of sheet piles is in retaining walls and cofferdams erected to enable permanent works to proceed. Normally, vibrating hammer, crane, and crawler drilling are used to establish sheet piles.

19.14.1.2 Steps of Construction

1. Construction of piles (large-diameter bored piles)
2. Driving of sheet piles
3. Dewatering to a level just below struts No. 1 and erection of struts No. 2, then to level below struts No. 2 to construct them and similarly struts No. 3
4. Excavate under water to the bottom level of plain concrete layer
5. Pour the plain concrete layer under water
6. Complete dewatering. The plain concrete layer together with the piles must resist the water uplift
7. Construct the R.C. pile cap
8. Construct the pier to a level above the water level
9. Remove the sheet piles

Figures 19.57 through 19.60 illustrate the steps of construction of the bridge foundations using sheet piles.

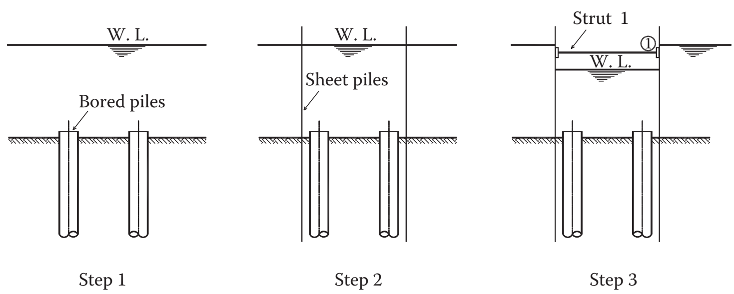


FIGURE 19.57 Steps 1, 2, and 3 of construction stages.

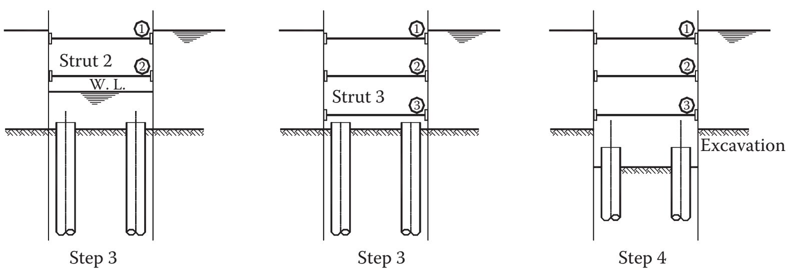


FIGURE 19.58 Steps 3 and 4 of construction stages.

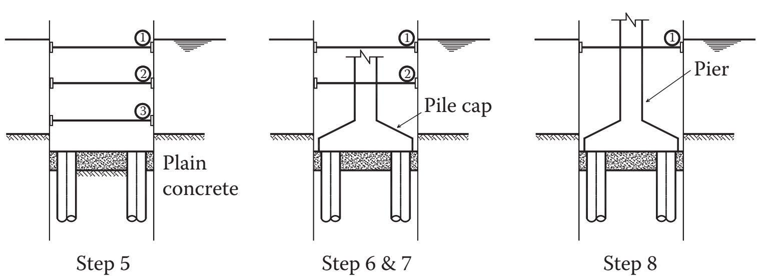


FIGURE 19.59 Steps 5, 6, 7, and 8 of construction stages.

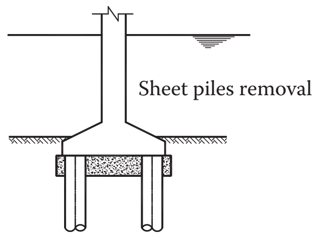


FIGURE 19.60 Step 9 of construction stages.

19.14.2 Submerging System for Construction of Pile Caps under Water

19.14.2.1 General

This method introduces the Egyptian experience in adapting lift-slab technology to construct submerged pile caps on the Nile River. Being submerged, these caps cause no obstruction to the ships moving through the river and thus provide a wide and clear waterway. Pile caps are constructed partly above water level, sunk to place, and monolithically completed with the pile group at the underwater permanent position. The novelty in the adapted lift-slab method is represented by the constructor-optional assembly and the use of a temporary construction system. The purpose was to work most of the time above water in dry and safe conditions and eliminate the need for cofferdams. This method was used to construct pile caps along the Nile River in Egypt during the 1990s. Table 19.3 presents a list of bridges where this technology was used.

19.14.2.2 Steps of Construction

1. Trimming tops of casings to provide equally protruding parts of about 50 cm above the highest water level. Cleaning out the inside of the casings down to the surface of pile concrete and recording racked and bulged casings where bigger clearances must be provided between casing and pile caps to attain smooth sinking.
2. To fix temporary supporting frames, plain concrete with formed 120-cm-deep cylindrical grooves are powered in the casings as shown in Figure 19.61.
3. The casing is ready in this stage to receive the temporary platform and pile cap formwork.
4. Steel I beams are stacked and welded to the casings to form the main casings of the main girders of the temporary platform. Secondary beams are stacked perpendicular to the main girders as shown in Figure 19.61.
5. Erection of the bottom formwork and the side formwork to form the horizontal and vertical dimensions of the pile cap as shown in Figure 19.61.
6. Cylindrical forms with open bottom and top are formed to intercept holes above the casings. The height of these forms is the same as the height of the cap bottom to be powered at this temporary position and equals the thickness of the bottom reinforcement of the cap plus top and bottom concrete covers. These cylindrical forms should be coaxial with the casings (Figure 19.61).
7. A 6-mm-thick washer is needed to capture the sealant necessary to seat the space between the casing and the pile cap is hung on a supporting angle. This circular support angle is installed in the bottom formwork at this stage to form the section as shown in Figure 19.62.
8. Temporary supporting frames are inserted in the grooves of the casings. The verticality is adjusted and the frames are fixed by powering plain concrete around the legs. This supporting frame is composed of a prop that supports two perpendicular I-beams as shown in Figure 19.63.
9. Concurrent with erecting frames, the steel reinforcement of the bottom and sides of the pile cap is fixed in place. The reinforcement of the remaining part of the cap is to be completed before sinking is fixed in place as shown in Figure 19.63.

TABLE 19.3 List of Bridges over the Nile Constructed Using Submerged Pile Caps

Number	Bridge Name	Opening Date	Length (m)	Width (m)	Number of Spans	Navigable Span (m)
1	Rod El-Farag Bridge	1990	500	36	9	130
2	Luxor Bridge	1997	744	22	18	84
3	El-Moneeb Bridge	1998	1700	42	26	150
4	Mit Ghamr-Zifta Bridge	1999	587	21	11	84
5	Sherbeen Bridge	1999	880	21	23	90

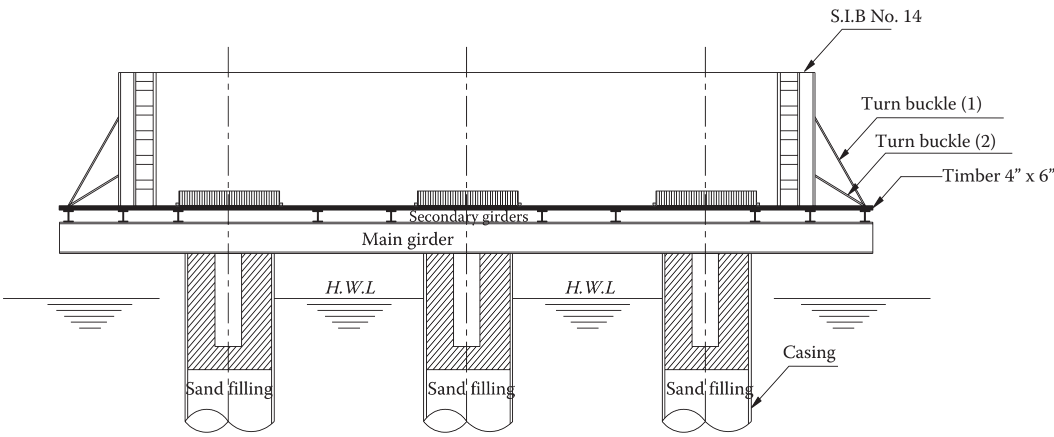


FIGURE 19.61 Preparation of temporary system of casting and sinking the pile cap (steps 1–6).

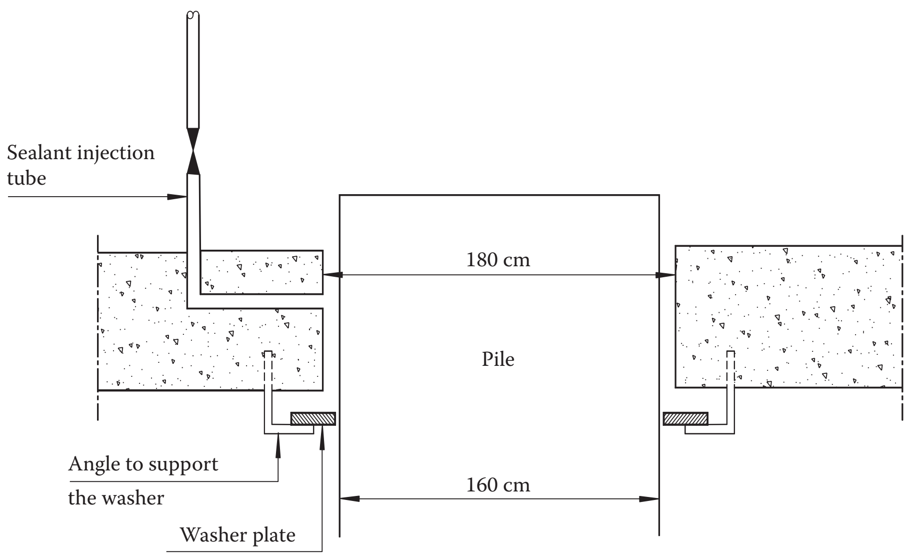


FIGURE 19.62 Details of sealing system of space between pile casing and pile cap (step 7).

10. The bottom of the pile cap is casted and concrete is powered in the space between the cylindrical forms with depth equal to the depth of the cylindrical forms. After the concrete sets, the cylindrical forms are dismantled and other vertically segmented cylindrical steel forms are used to form working rooms of height equal to the full height of the pile cap and diameter of about 250 cm.

11. The reinforcement of the space between the working rooms is completed, extended reinforcements are left to provide the lap with the reinforcement that will be fixed later in the space of the working rooms.
12. At this stage, all the casting works are completed before the sinking process as shown in Figure 19.64.
13. After setting, the forms of the fender are stripped off as well as the side forms of the pile cap as shown in Figure 19.63. Any insulation work for the pile cap and the fender is performed at this stage.
14. At this stage, the lifting equipment is installed. Screw bars that are fixed at the bottom of the pile cap are extended with additional ones. Hydraulic jacks are mounted on the temporary supporting frame and attached to the corresponding screw bars. Connections between jacks and hydraulic pumps are installed.

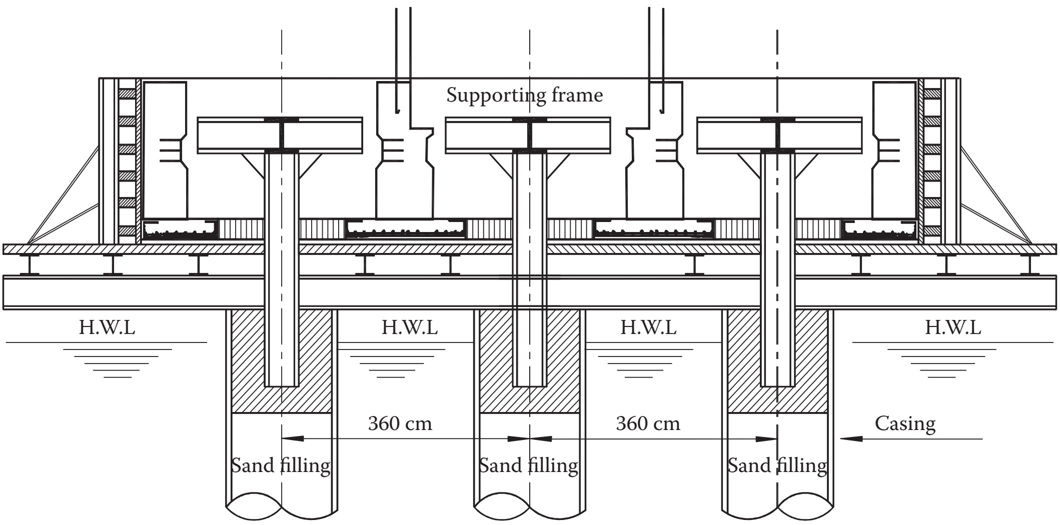


FIGURE 19.63 Erecting frames and fixing steel reinforcement of bottom and sides of pile cap steps 8 and 9.

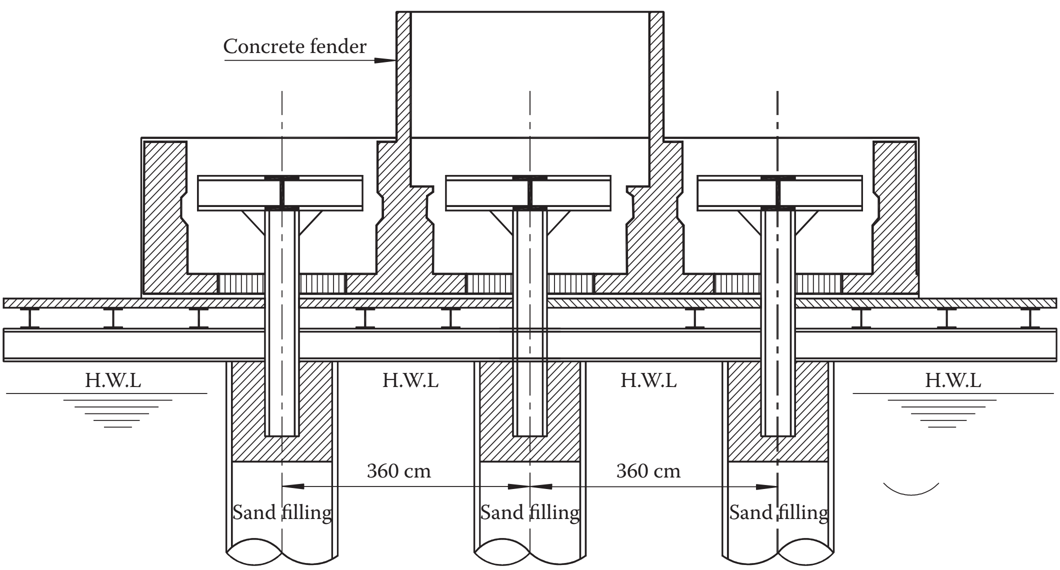


FIGURE 19.64 Pouring the bottom, forming working rooms, and pouring around working rooms (step 12).

15. A controlled system is installed between the hydraulic pump and the jack to provide lifting and lowering at the same rate.
16. The jacks are operated to lift the cap about 50 cm and the temporary platform and bottom formwork of the cap are stripped off. At this stage, the pile cap is completely suspended on the supporting frames (Figure 19.65).
17. The sinking process is started by lowering the pile cap slowly until the top of the pile cap is about 50 cm above water surface. The sinking process proceeds until the bottom of the cap reaches the final level as shown in Figure 19.66.

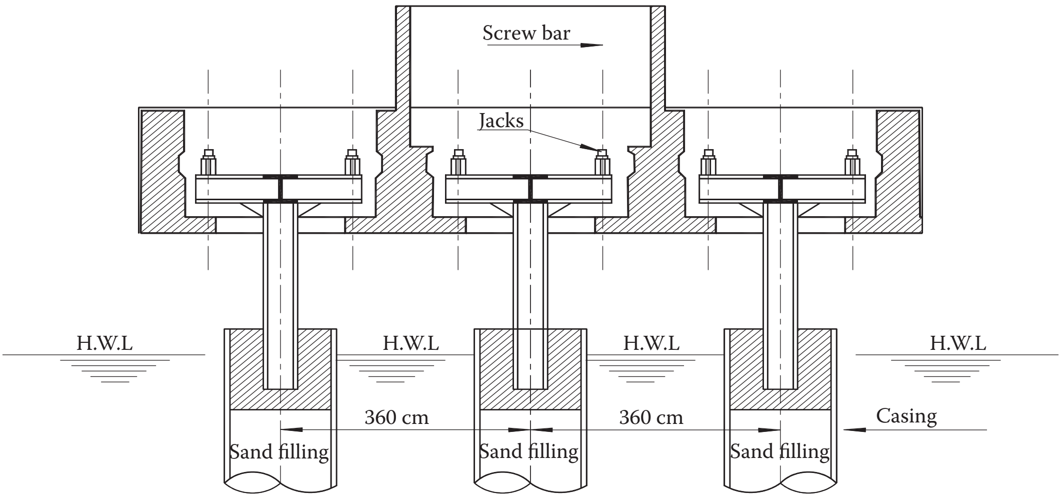


FIGURE 19.65 Lifting the pile cap, stripping off side and bottom formwork, and taking apart the temporary platform (step 16).

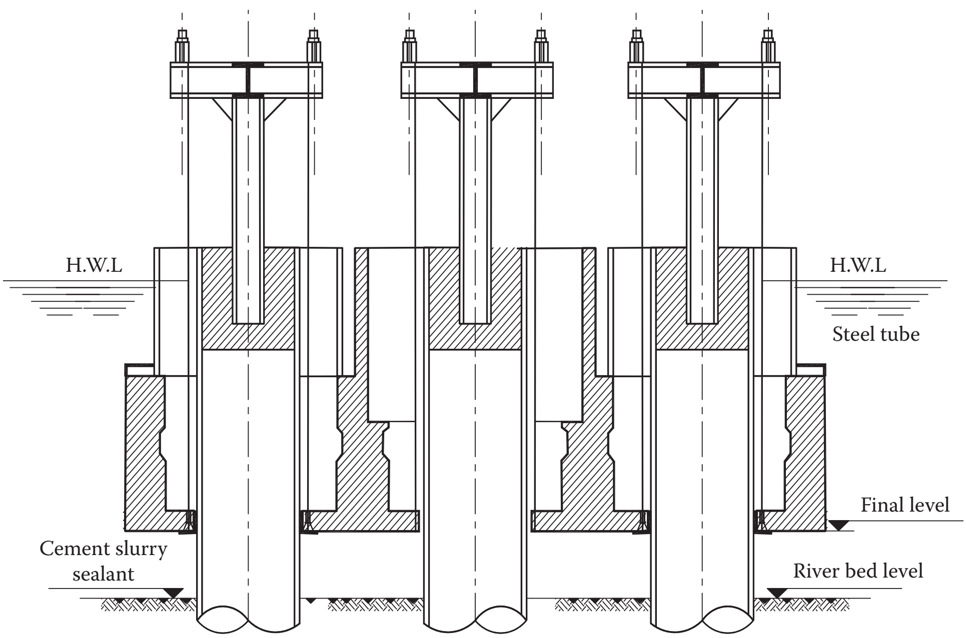


FIGURE 19.66 Sinking the pile cap to permanent position (step 17).

18. At this stage, the construction proceeds at the permanent position of the pile cap. Starting with the piles without frames, the steel casing and concrete are removed down to the surface of the cap bottom layer. The protruding steel bars are cleaned. The concrete surface of the working rooms is roughened to ensure enough cohesion and bond. The lap bars of reinforcement of the middle layers are uncovered, stretched, and cleaned. Extra horizontal reinforcement and the column reinforcement are fixed in place. Then these working rooms are filled with concrete.
19. For piles with supporting frames, these frames are dismantled. The work of the previous step is repeated but at stages to ensure safe transmission of the cap load to the piles. First, the working room inside the fender is prepared with reinforcement of the column as shown in Figure 19.67. Subsequently, the work at the other working rooms proceeds until the whole pile cap is completed.
20. The last step is to erect the carpentry of the column, fixing the reinforcement and pouring the concrete as shown in Figure 19.68.

Figures 19.69 through 19.71 show the adoption of this method during the construction of the pile caps of El-Moneeb Bridge.

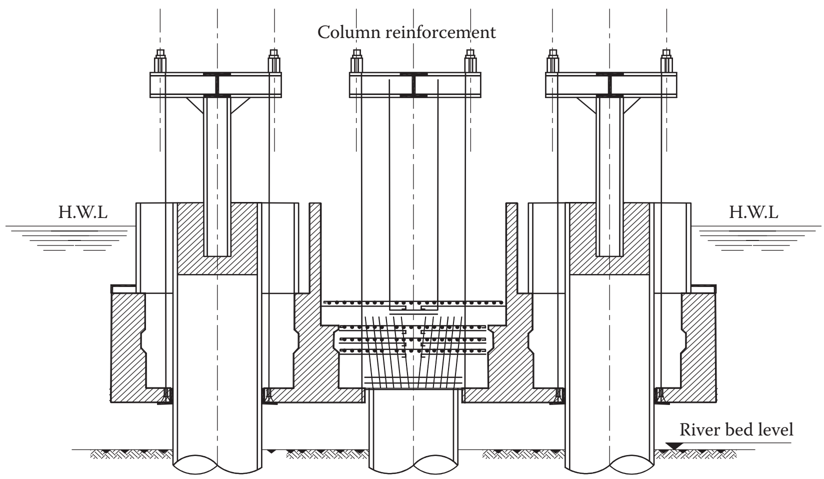


FIGURE 19.67 Pouring of some piles monolithically with pile cap (step 19).

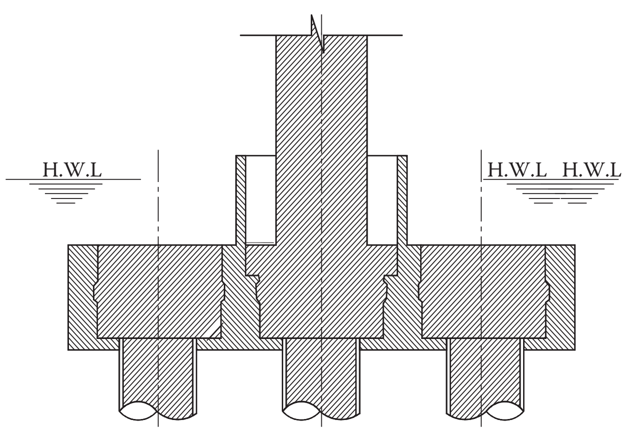


FIGURE 19.68 Pile cap after finishing the pouring of the concrete (step 20).

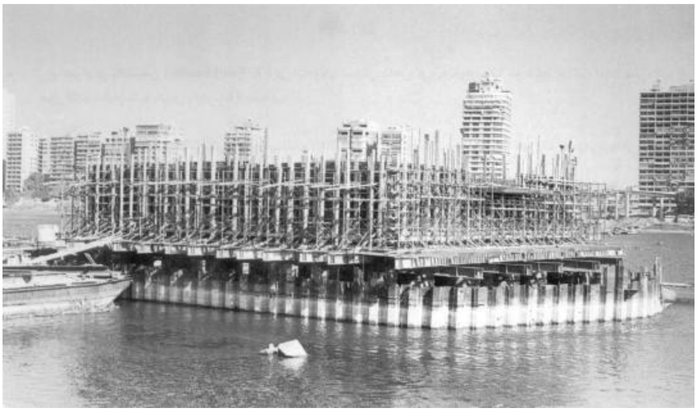


FIGURE 19.69 Preparation of the formwork for casting (El-Moneeb Bridge).

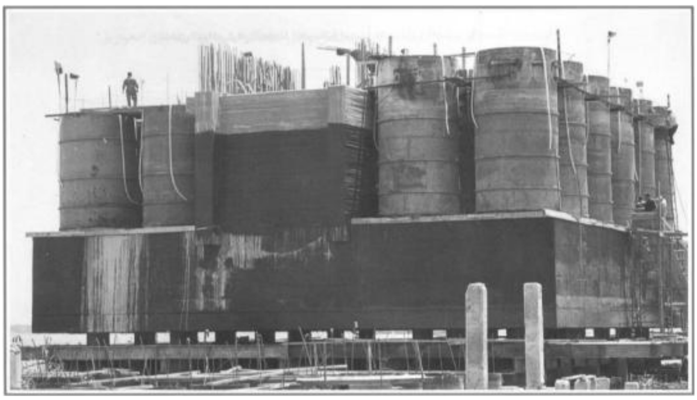


FIGURE 19.70 After pouring the bottom, sides, and forming the working rooms (El-Moneeb Bridge).

FIGURE 19.71 Erected steel frames and hydraulic jacks used for lifting the cap (El-Moneeb Bridge).

19.14.3 Foundation Construction Using Caissons

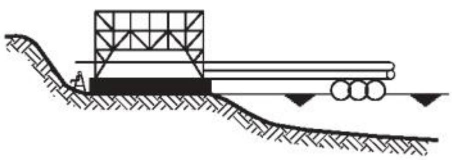
19.14.3.1 General

A caisson is a watertight retaining structure used to work on the foundations of a bridge pier for the construction of a concrete dam or for the repair of ships. These are constructed such that the water can be pumped out, keeping the working environment dry. When piers are to be built using an open caisson and it is not practical to reach suitable soil, friction pilings may be driven to form a suitable subfoundation. These piles are connected by a foundation pad upon which the column pier is erected.

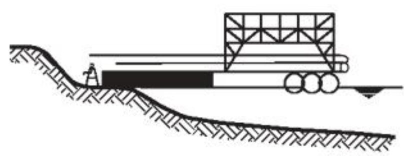
19.14.3.2 Steps of Construction

1. Caisson at the shore
2. Moving the caisson to the slipway
3. Inclining the slipway to float the caisson
4. Tugboats moving the caisson into position
5. Fixing the caisson in position by means of floating tubes and concrete block
6. Gradual concrete sinking and adding new steel plates to the caisson
7. Caisson resting on river bed and erection of locks
8. Excavating to the foundation level
9. Reaching foundation level and filling the working with concrete

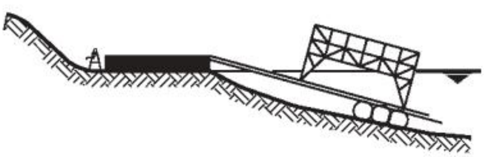
Figure 19.72 illustrates the steps of construction of the bridge foundations using caissons.



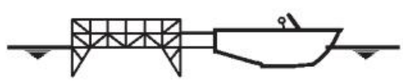
1. Caisson at the Nile shore



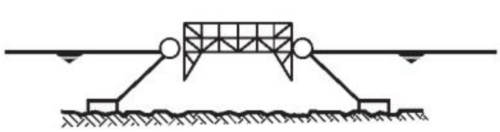
2. Moving the caisson to the slipway



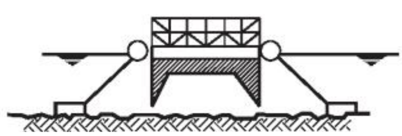
3. Inclining the slipway to float the caisson



4. Tugboats moving the caisson into position



5. Fixing the caisson in position by means of floating tubes and concrete blocked



6. Gradual concrete sinking and adding new steel plates to the caisson

FIGURE 19.72 Steps of construction of bridge foundations using caissons.

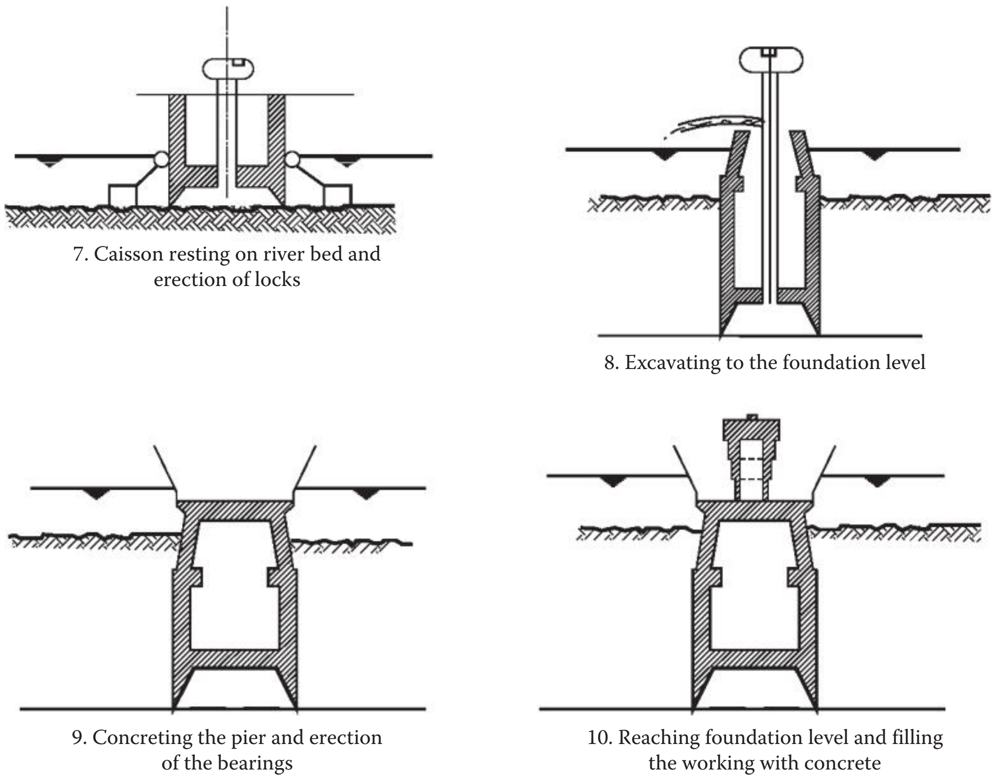


FIGURE 19.72 (*Contiuned*) Steps of construction of bridge foundations using caissons.

References

Basha, I. M. 1991. "The Construction of Ghamra Bridge Superstructure Using Launching Truss," *Proceedings of Al-Azhar Engineering 2nd International Conference*, Al-Azhar University, Cairo, Egypt.

Benaim, R. 2008. *The Design of Prestressed Concrete Bridges: Concepts and Principles*, CRC Press, Boca Raton, FL, 608pp.

Combault, J. 2008. Conceptual Design of Bridges, Short and Medium Span Bridges, *1st International Symposium on Bridges and Large Structures*, Brazil, May 2–8, 2008.

fib. 2000. *Guidance for Good Bridge Design*, Bulletin 9, International Federation for Structural Concrete.

JICA. 1991. *Course Notes* (*Page 2*), *Prestressed Concrete Bridge Construction Methods*, Japan International Cooperation Agency, Tokyo, Japan.

Jungrwirth, D. J. 1998. Prestressed Concrete Bridges; Flop or Technical Progress, FIP Structural Concrete 1994–1998, German Group of FIP, FIP Congress 1998.

Liebenberg, A. C. 1992. *Concrete Bridges: Design and Construction*, Longman Scientific & Technical, Longman Group UK Limited, Harlow, UK.

Nakai, H., and C. H. Yoo. 1988. *Analysis and Design of Curved Steel Bridges*, McGraw-Hill, New York, NY, 673pp.

Rajagopalan, N. 2006. *Bridge Superstructure*, Alpha Science International Ltd., UK.

VSL. 2013. *Bridge Construction Partner*, VSL International, Köniz, Switzerland. http://www.vsl.com/images/brochures/bridges/bridge-construction-partner/view/index.html.

Youssef, M. A., C. J. Anumba, and T. Thorpe. 2005. "Intelligent Selection Of Concrete Bridge Construction Methods In Egypt," *Proceedings of the ASCE International Conference on Computing in Civil Engineering*, July 12–15, American Society of Civil Engineers, Cancun, Mexico.

Bibliography

AASHTO. 2012. *AASHTO LRFD Bridge Construction Specifications*, 3rd Edition with 2012 Interim, American Association of State and Highway Transportation Officials, Washington, DC.

AASHTO/NSBA. 2003. *Guidelines for Design for Constructability*, G12.1-2003, AASHTO/NSBA Steel Bridge Collaboration, AASHTO, Washington, DC.

AASHTO/NSBA. 2007. *Steel Bridge Erection Guide Specification*, S10.1-2007, AASHTO/NSBA Steel Bridge Collaboration, AASHTO, Washington, DC.

Abhyankar, V. G. 2011. "Bridge Erection Techniques and Their Influence on Permanent Designs," *National Workshop at COEP, Pune on Innovation in Bridge Engineering—15–16th Oct'2011*. Under Late Shri S. B. Joshi Memorial Activity Bridge.

Abrahams, M. J. 2000. "Movable Bridges, Past, Present and Future," *Bridge Engineering Conference*, organized by Egyptian Society of Engineers (ESE)/ Civil & Egyptian Groups of IABSE and fib, March 26–30, Sharm El Sheikh, Egypt.

ACI 345R-11. 2011. *Guide for Concrete Highway Bridge Deck Construction*, American Concrete Institute, Farmington Hills, MI.

André, J., R. Beale, and A. Baptista. 2012. "Bridge Construction Equipment: An Overview of the Existing Design Guidance," *Structural Engineering International*, 22(3), pp. 365–379.

Arab Contractors. 1998a. *Cantilever Carriage System, Engineer's Guide in Construction*, Bridges and Specialized Structures Department, The Arab Contractors Press Osman Ahmed Osman & Co., Cairo, Egypt.

Arab Contractors. 1998b. *Deck Pushing System, Engineer's Guide in Construction*, Bridges and Specialized Structures Department, The Arab Contractors Press Osman Ahmed Osman & Co., Cairo, Egypt.

Arab Contractors. 1998c. *Lowering System, Engineer's Guide in Construction*, Bridges and Specialized Structures Department, The Arab Contractors Press Osman Ahmed Osman & Co., Cairo, Egypt.

Arab Contractors Osman Ahmed Osman & Co. 2013. *Bridges and Flyovers*, http://www.arabcont.com/English/projects/Domestic_Projects.aspx?sec_id=4&subsec_id=1.

ASBI. June 2008. *Construction Practices Handbook for Concrete Segmental and Cable-Supported Bridges*, 2nd Edition, American Segmental Bridge Institute, Buda, TX.

Attar, A. G. 2002. *Dynamic Testing of Qanater Railway Bridge*, Faculty of Engineering, Cairo University.

Bakhoum, M. M., G. Morcous, M. El-Said, and M. Taha. 1998. "Estimate of Quantities and Cost of Prestressed Concrete Bridges over the Nile in Egypt," *Journal of Egyptian Society of Engineers/Civil*, 37(4), 17–32.

Basha, I. M. 1989a. "Bridge Construction in Egypt," *Arab Roads Journal*, 2nd Edition.

Basha, I. M. 1989b. "Bridge Construction Systems Evacuation," *Arab Roads Journal*, 2nd Edition.

Basha, I. M. 1997. "Innovative Site Layout Approach for Al-Warrak Bridge Project," *Proceedings of Al-Azhar Engineering 5th International Conference*, Al-Azhar University, Cairo, Egypt.

Basha, I. M., and A. A. Gab-allah. 1991. "Value Engineering in Egyptian Bridge Construction," *Journal of Construction Engineering and Management*, ASCE, 117(3), 303–401.

Bernard-Gely, A., and J. A. Calgaro, 1994. *Conception des Ponts* (Design of Bridges), Presses de l'Ecole Nationale des Ponts et Chaussées ENPC, 361pp.

Bindra, S. P., and K. Bindra. 1976. *Elements of Bridge, Tunnel & Railway Engineering*, Dhanpat Rai and Sons, Delhi, India.

Blank, S. A., M. M. Blank, and H. Kondazi. 2014. "Chapter 3: Concrete Bridge Construction," *Bridge Engineering Handbook, Second Edition: Construction and Maintenance*, eds. W. F. Chen and L. Duan, CRC Press, Boca Raton, FL.

Braun, C. 2011. "Structures Entailing Bridge Bearings and Expansion: The Differential Construction Method in Bridge Construction," *BRÜCKENBAU*, pp. 40–43.

Calgaro, J. A., and M. Virlogeux. 1987. *Projet et Construction des Ponts*, Presses de l'École Nationale des Ponts et Chaussées, Paris, France.

Cardwell, S. 2010. "Bridge deck types and construction methods," *IABSE WG 6 Seminar State-of heart Bridge Deck Erection; Safe and Efficient use of Special Equipment*, International Association for Bridge and Structural Engineering IABSE, Singapore.

CBDG. 2005a. *Fast Construction of Concrete Bridges*, Technical Guide No. 5, Concrete Bridge Development Group, Surry, UK.

CBDG. 2005b. *Fast Construction Segmental and Launched Bridges*, by Bourne, S., Technical Paper 9, Concrete Bridge Development Group, Surry, UK.

Chen, B., and J. Radić., eds. 2009. "Construction of Arch Bridges," *Proceedings of the 2nd Chinese-Croatian Joint Colloquium*, Fuzhou University College of Civil Engineering & University of Zagreb Faculty of Civil Engineering, Fuzhou, China, October 5–9, 2009.

Chen, W. F., and E. M. Liu. 2005. *Handbook of Structural Engineering*, 2nd Edition, Taylor & Francis, Boca Raton, FL, 2020pp.

Chen, W. F., and L. Duan, eds. 2014. *Bridge Engineering Handbook, Second Edition: Construction and Maintenance*, CRC Press, Berkeley, CA.

CIRIA R155. 1996. *Bridges—Design for Improved Buildability*, CIRIA, London, UK, 162pp.

Combault, J. 2009. *40 years of Experience Segmental Bridge Construction*, 33rd IABSE Symposium, Bangkok, Thailand.

Durkee, J. 2014. "Chapter 1: Steel Bridge Construction," *Bridge Engineering Handbook, Second Edition: Construction and Maintenance*, eds. W. F. Chen and L. Duan, CRC Press, Boca Raton, FL.

Eaton, R. A., and C. Boring. 2011. Design for Constructability, *Steel Bridge Design Handbook*, Volume 11, FHWA-IF-12-052-Vol.11, http://www.fhwa.dot.gov/bridge/steel/pubs/if12052/volume11.pdf.

Elazouni, A. M., and M. E. A. El-Razek. 2000. "Adapting Lift-Slab Technology to Construct Submerged Pile Caps," *Journal of Construction Engineering and Management*, 126(2), 140–157.

El-Razek, M. E. A., and I. M. Basha. 2001. "Constructability Improvement of Bridges Using Stepping Formwork," *Journal of Construction Engineering and Management*, 127(3), 206–213.

EN 1090-1:2009 + A1:2011. *Execution of steel structures and aluminum structures 3 parts*, European Committee for Standardization, CEN, Brussels, Belgium.

EN 12699:2000. *Execution of special geotechnical work—Displacement piles*, European Committee for Standardization, CEN, Brussels, Belgium.

EN 12812:2008. *Falsework. Performance Requirements and General Design*. European Committee for Standardization, CEN, Brussels, Belgium.

EN 13670:2009. *Execution of concrete structures*, European Committee for Standardization, CEN, Brussels, Belgium.

EN 14475:2006/AC:2006. *Execution of special geotechnical works—Reinforced fill*, European Committee for Standardization, CEN, Brussels, Belgium.

EN 1536:2010. *Execution of special geotechnical work—Bored piles*, European Committee for Standardization, CEN, Brussels, Belgium.

Fédération Internationale de la Précontrainte. 1998. *Structural Concrete 1994–1998*, Deutscher Beton-Verein E.V., Berlin, Germany, 153pp.

fib. 2009. *Formwork and Falsework for Heavy Construction: Guide to Good Practice*, Bulletin 48, International Federation for Structural Concrete.

Garrell, S. 2012. "How to Estimate the Cost of Bridge Construction," *Estimating Today Journal*, www.aspenational.org.

Gerwick, B. C. Jr. 1997. *Construction of Prestressed Concrete Structures*, 2nd edition, John Wiley & Sons, Inc., Toronto, Ontario, Canada. 616pp.

Gerwick, B. C. Jr. 2014. "Chapter 6: Substructures of Major Overwater Bridges," *Bridge Engineering Handbook, Second Edition: Construction and Maintenance*, eds. W. F. Chen and L. Duan, CRC Press, Boca Raton, FL.

Gimising, N. J. (editor). 1998. *East Bridge*. A/S Storebaelt Publications, Copenhagen, Denmark.

Grattesat, G. 1981. *Conception des Ponts*, Cours de l'école Nationale des Ponts et Chaussées, France.

Hambly, B. C. 1979. *Bridge Foundations and Substructures*, Building Research Establishment Report, HM Stationery Office, London, UK.

Harris, H. 2013. *Modern Construction and Ground Engineering Equipment and Methods*, Prentice Hall, West Sussex, UK, 632pp.

Hewson, N. R. 2003. *Prestressed Concrete Bridges: Design and Construction*, Thomas Telford, London, UK, 371pp.

Holst, K. H. 1996. *Bridges of reinforced concrete and prestressed concrete: Design, construction and calculation*, 5th ed., Auflage: 3, Ernst & Sohn, Berlin, Germany, (in German).

Hussain, N., K. F. Hansen, and S. Kite. 2010. *Stonecutters Bridge Hong Kong: Design and Construction*. Ove Arup Partners Hong Kong Ltd, Hong Kong.

Hussain, N. 2013. *Guidelines for Design Competitions for Bridges*, IABSE, International Association for Bridge and Structural Engineers, Zurich, Switzerland.

IABSE WG-6. 2010. *State-of-the-Art Bridge Deck Erection: Safe and Efficient Use of Special Equipment*. IABSE WG-6 Seminar, Singapore, November 15–16, 2010, Organized by IABSE Working Group 6 on Bridge Construction Equipment.

ISTED. 1987. *Le Savoir Faire Français en Matière D'ouvrages D'art* [French Know-How in Bridge Building], Institut des sciences et des techniques de l'equipement et de l'environnement pour le developpement, France, 365pp.

JICA. 1991a. *Course Note: Prestressed Concrete Bridge Construction Method*, Japan International Cooperation Agency JICA, TITC-J R, 91–76, Japan.

JICA. 1991b. *Course Note: Textbook for the Group Training Course in Bridge Engineering II*, Japan International Cooperation Agency JICA, Vol. I, TITC-J R, Japan.

Leonhardt, F. 1979. *Vorlesungen über Massivbau—Teil 6—Grundlagen des Massivbrückenbaues in German*, Springer-Verlag, Berlin, Germany.

Leonhardt, F. 1982. *Brucken/Bridges*, The Architectural Press, London, UK.

Leonhardt, F. 1984. *Bridges*, MIT Press—Technology & Engineering, Cambridge, MA, 307pp.

Libby, J. R. 2007. *Modern Prestressed Concrete*, 3rd ed., CBS Publishers & Distributors Pvt. Ltd., New Delhi, India.

Libby, J. R., and N. D. Perkins. 1977. *Modern Prestressed Concrete Highway Bridge Superstructures: Design Principles and Construction Methods*, Grantville Publishing Company, San Diego, CA, 254pp.

Liu, H. X., M. F. F. Siu, R. Ekyalimpa, and L. Ming. 2012. Simulation of mobile falsework utilization methods in bridge construction, *Proceedings of the 2012 WSC Winter Simulation Conference*, Berlin, Germany, eds. O. Rose, A. M. Uhrmacher, Berlin, Germany, December 9–12, 2012, pp. 1–13.

Mahlab, I. 2006. *New Bridge Construction Techniques—Case Studies, International Conference on Bridge Management Systems Monitoring, Assessment and Rehabilitation*, Housing & Building National Research Center, HBRC, Cairo, Egypt.

Mallela, J. 2012. *Accelerated Construction Techniques ABC and PCPS*, Denver, CO, http://www.ops.fhwa.dot.gov/wz/p2p/arw/p1_mallela.htm.

Marzouk, M., H. Said, and M. El-Said. 2008. "Special-Purpose Simulation Model for Balanced Cantilever Bridges," *Journal of Bridge Engineering*, 13(2), 122–131.

Mathivat, J. 1983. *The Cantilever Construction of Prestressed Concrete Bridges*, John Wiley & Sons Ltd., New York, NY, 352pp.

Menn, C. 1989. *Prestressed Concrete Bridges*, Birkhauser; Springer-Verlag, Wien.

Meyer, M. E. 2008. "State-of-Art Bridge Deck Construction—The Challenge to Choose the Right Method," *Innovative World of Concrete, 4th International Conference & Exhibition*, ICI, Delhi, 2008.

Miller, R. A. 2005. *Innovative Bridge Design/Construction Techniques to Expedite Construction*, Ohio Department of Transportation Office of Research and Development and the U.S. Department of Transportation Federal Highway Administration, Columbus, OH.

Mitchell, A. J. 1983. "Caisson Foundations," *Handbook of Structural Concrete*, McGraw-Hill, New York, NY.

Mitsubishi Heavy Industries Bridge & Steel Structures Engineering Co., Ltd. 2012. "Cutting-Edge Technologies to Shorten Bridge Construction Time: The Rapid Launching and Jack-Down Method for the Erection of 2700 tons of Steel Girders," *Mitsubishi Heavy Industries Technical Review*, Vol. 49, No. 4, December 2012, www.mhi.co.jp/technology/review/pdf/e494/e494054.pdf.

Moharram, A. Jr. 2000. "High-Strength Concrete Bridges Built by Cantilevering," *Proceedings of the ESE Bridge Engineering Conference*, Egyptian Society of Engineers (ESE)/Civil & Egyptian Groups of IABSE and fib, Sharm El Sheikh, Egypt, March 26–30.

Muller, J. 1975. *Ten Years of Experience in Precast Segmental Construction: A Special Report*, Prestressed Concrete Institute, Chicago, IL, 35pp.

Nasr General Contracting H. M. Allam. 2013. *Bridges and Flyovers*. http://www.ngcc-allam.com.eg/bridg/project_Emain.html.

Nitschke, J. 2010. "Form-Traveller Systems," *IABSE WG 6 Seminar State-of-the-art Bridge Deck Erection; Safe and Efficient use of Special Equipment*, International Association for Bridge and Structural Engineering IABSE, Singapore.

O'Connor, C. 1971. *Design of Bridge Superstructures*, Wiley, Chichester, New York.

Oliver Rose, Adelinde M. Uhrmacher (eds.): *Winter Simulation Conference*, WSC '12, Berlin, Germany, December 9-12, 2012. WSC 2012.

Ostenfeld, K. H. 2010. "An Integrated Multidisciplinary Approach to Design of Major Fixed Links," *IABSE Symposium*, Venice, pp. 1–3232.

Palmer, A. M. 2006. "Fundamentals of Launching a Precast Concrete Segmental Operation for Bridge Construction Projects," *PCI Journal*, 51(3), 32–44.

Pan, N. F. 2008. "Fuzzy AHP approach for Selecting the Suitable Bridge Construction Method," *Automation in Construction,* 17(8), 958–965.

Parke, G., and N. Hewson. 2008. *ICE Manual of Bridge Engineering*, 2nd Edition, Thomas Telford Ltd, London, UK, 704pp.

PCI. 1997. *Precast/Prestressed Concrete, Bridge Design Manual*, PCI, MNL-133-97, Chapter 3, Fabrication and Construction, Precast/Prestressedconcrete Institute, PCI, Chicago, IL.

PCI. 2006. *Guidelines for Accelerated Bridge Construction Using Precast/Prestressed Concrete Components*. PCI Northeast Bridge Technical Committee, Precast/Prestressed Concrete Institute, North East, Belmont, MA http://www.pcine.org/index.cfm/resources/bridge/Accelerated_Bridge_Construction.

Pearce, M., and R. Jobson. 2002. *Bridge Builders*. Wiley-Academy, West Sussex, UK.

Pelke, E. 2006. The Development of the Prestressed Concrete Bridge in Germany after World War II. *Proceedings Second International Congress on Construction History Society Vol. III*, eds. M. Dunkeld, J. Cambell, H. Louw, B. Addis, R. Thome, Construction History Society, Cambridge, UK, pp. 2469–2492.

Podolny, W. Jr., and J. M. Muller. 1982. *Construction and Design of Prestressed Concrete Segmental Bridges*, Wiley-Interscience, New York, NY, 561pp.

Radic, J., G. Puz, and Z. Zderic. 2008. *RE Construction of Bridges in Croatia*, Velika Gorica, Academia scientiarum et artium croatica, Zagreb, Croatia.

Ratay, R. T. 1984. *Handbook of Temporary Structures in Construction*, McGraw-Hill, New York, NY.

Roberts, C. L., J. E. Breen, and M. E. Kreger. 1993. *Measurement Based Revisions for Segmental Bridge Design and Construction Criteria*. Research Report 1234-3F, Project 0-1234, CTR 0-1234-3F, Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin, Austin, TX.

Rosignoli, M. 2002. *Bridge Launching*. Thomas Telford Ltd, Technology & Engineering, London, UK, 342pp.

Rosignoli, M. Winter 2010. "Self-Launching Erection Machines for Precast Concrete Bridges," *PCI Journal*, 55(1), 36–57.

Rosignoli, M. 2013. *Bridge Construction Equipment*, 1st Edition, ICE Publishing, UK.

Said, H., M. Marzouk, and M. El-Said. 2009a. "Application of Computer Simulation to Bridge Deck Construction: Case Study," *Automation in Construction*, 18(4), 377–385.

Said, H., M. Marzouk, and M. El-Said. 2009b. "Framework for Multiobjective Optimization of Launching Girder Bridges," *Journal of Construction Engineering and Management*, 135(8), 791–800.

Samaan, S. 1994. *Course Notes on Methods of Construction of Bridges*, Construction Engineering, Graduate Studies, Faculty of Eng., Structural Eng. Dept., Cairo University, Cairo, Egypt.

Schlaich, J. 2000. "Conceptual Design of Bridges—More Variety!" *Bridge Engineering Conference*, organized by Egyptian Society of Engineers (ESE)/Civil & Egyptian Groups of IABSE and fib, Sharm El Sheikh, Egypt, March 26–30.

Schlaich, J., and H. Scheef, 1982. "Concrete Box Girder Bridges," *Structural Engineering Document, SED 1, IABSE*, International Association for Bridge and Structural Engineering, IABSE/AIPC/IVBH, Zurich, Switzerland.

SCI. 2010. Steel Bridge Group. *Guidance notes on best practice in steel bridge construction*, eds. C. R. Hendy, and D. C. Iles, The Steel Construction Institute, Fifth Issue P185/5, TATA Steel. Steel Construction Institute, Ascot, UK, SCI.

SDR/FHWA. 2005. *Prefabricated Steel Bridge Systems*, FHWA Solicitation No. DTFH61-03-R-00113, Prepared by SDR Engineering Consultants, Inc, http://www.fhwa.dot.gov/bridge/prefab/psbsreport.pdf.

SEI IABSE 4/. November 2011. "Special Topic: Bridge Erection Techniques and Construction Equipment," *Structural Engineering International, International Association for Bridge and Structural Engineering*, 21(4), 392–456.

SETRA. 2003. *Ponts en béton précontraint construits par encorbellements successifs: Guide de conception*, La Documentation Française, Paris, France, 271pp.

Singh, P. R., M. Schueller, and M. Bowser. 2010. "Innovative Steel Bridge Erection Techniques," *TAC Conference*, Halifax, Canada.

Tomlinson, M. J. 2001. *Foundation Design and Construction*, 7th Edition, Prentice Hall, Essex, UK, 584pp.

Tordoff, D. 1985. *Steel Bridges*, Steel Construction Institute, UK, 44pp.

Trayner, D. 2007. *Bridge Construction Methods*, Concrete Institute of Australia, http://www.ptia.org.au/Documents/Bridge%20Construction%20Methods%20Aug%202007%20rev%2000.pdf.

Troyano, L. F. 2003. *Bridge Engineering: A Global Perspective*, Thomas Telford Ltd, London, UK, 775pp.

Victor, J. 1980. *Essentials of Bridge Engineering*, Oxford & IBH Publishing, New Delhi, India.

Vion, P., and E. Bouchon. "Prestressed concrete bridges built by the cantilever method, Design and stability during erection," *New Delhi fib Symposium on Segmental construction in concrete*, SETRA, November 26–29, http://www.setra.equipement.gouv.fr/IMG/pdf/pcbridges_cantilevermethod.pdf.

Virlogeux, M. August 1, 1993. "Normandie Bridge: Design and Construction," *Proceedings of the ICE—Structures and Buildings*, 99(3), 281–302.

Von Svensson, H. 2011. *Schrägkabelbrücken: 40 Jahre Erfahrung weltweit*, Ernst & Sohn, Germany.

VSL. 1977. *Incremental Launching Method Prestressed Concrete Bridge Construction*, VSL International LTD, Berne, Switzerland.

Whitney, C. S. 2003. *Bridges of the World: Their Design and Construction*, Dover Publications, Mineola, NY, 368pp.

Wittfoht, H. 1984. *Building Bridges: History, Technology, Construction*, Bau Verlag u. Technik, Beton-Verlag, Philadelphia, PA, 327pp.

Wittfoht, H. 1984. *Building Bridges: History, Technology, Construction*, Beton-Verlag, Berlin, Germany.

Wong, R. 2012. *Construction of Bridges, Division of Building Science and Technology*, City University of Hong Kong, Hong Kong,. http://personal.cityu.edu.hk/~bswmwong/pl/pdf/bridge_construction.pdf.

Youssef, M. A. M. 2006. *Intelligent Selection of Concrete Bridge Superstructure Construction Methods in Egypt*, Ph.D. Thesis, Loughborough University, Leicestershire, UK.

Relevant Websites

Websites for Bridge Equipment Construction Companies & Techniques

VSL_http://www.vsl.com/business-lines/construction/bridges.html

Freyssinet_http://www.freyssinet.com/appli/internet/w3fcom.nsf/ag_Creation_Page?OpenAgent&UNID=1B329863513BB0D8C12573AD003B723F&contexte=112&rubrique=activites&lang=en&font=small

ThyssenKrupp_http://www.constructionequipment.com/company/thyssenkrupp-safway-inc-0

BBR_http://www.bbrnetwork.com/index.php?id=66

OVM_http://www.ovm-mena.com/projects.html#egypt

PERI_http://peri.co.za/en/products.cfm/fuseaction/showproduct/product_ID/1059/app_id/8.cfm

DOKA _http://www.slideshare.net/doka_com/dokacompanypresentation-en

DSI_http://www.dsiamerica.com/company/product-index/construction.html

JASBC. Japan Association of Steel Bridge Construction_www.jasbc.or.jp/english

LIFTSLAB MISR_http://www.liftslab-eg.com/

NRSAS_http://www.nrsas.com/equipments/bb_technical.php

NRJS_http://en.nrsjs.com/products_list/pmcId=31e88d8d-6793-4781-adf5-97697b9579f4&comp_stats=comp-FrontPublic_breadCrumb01-008.html

REID_http://www.reids.co.nz/products/

STRUKTURAS_http://www.strukturas.com/bridge-building-equipment

AP_http://www.ap-bridge.com/index.php?option=com_content&view=article&id=19&Itemid=57&lang=en

FIP_http://www.fip-group.it/fip_ind_eng/mappa.html

DEAL_http://www.deal.it/equipment.asp

E.CRPCEC_CHINA_http://e.crpcec.com/tabid/1468/Default.aspx

SS_http://www.structuralsystemsafrica.com/services-aamp-technology/civil/bridge-construction-systems.html

Websites for Animations and Videos on Bridge Construction

ASBI (American Segmental Bridge Institute), Construction Precast Concrete Segmental Bridges

http://asbi-assoc.org/index.cfm/resources/videos

IABSE-eLearning_http://www.elearning-iabse.org/

IABSE-eLearning_Animation_http://www.elearning-iabse.org/MainPage.asp?cat=4

IABSE-eLearning_Video_http://www.elearning-iabse.org/MainPage.asp?cat=3

Websites on Accelerated Bridge Construction (ABC, PBES)

ABC_http://ops.fhwa.dot.gov/wz/construction/accelerated/index.htm

PBES_http://www.fhwa.dot.gov/everydaycounts/pdfs/summits/PBES_ABC_Current_St_of_Technology_presentation.pdf.

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