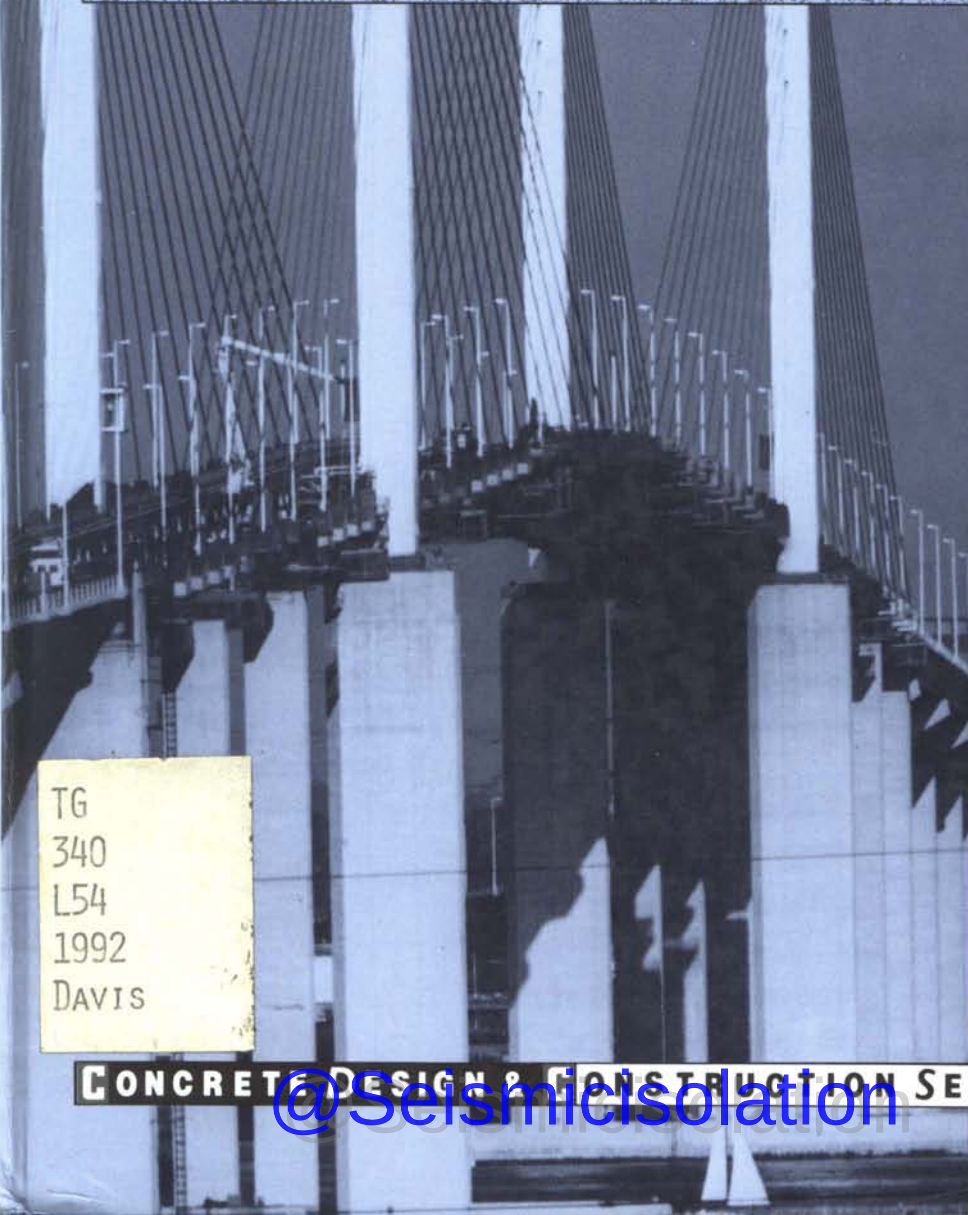
CONCRETE BRIDGES:

DESIGN AND CONSTRUCTION

A C LIEBENBERG



CONCRETE DESIGN & CONSTRUCTION SERIES

Concrete Bridges

Design and Construction

@Seismicisolation

Concrete Design and Construction Series

SERIES EDITORS

PROFESSOR F. K. KONG
University of Newcastle upon Tyne

EMERITUS PROFESSOR R. H. EVANS CBE
University of Leeds

OTHER TITLES IN THE SERIES

Concrete Radiation Shielding: Nuclear Physics, Concrete Properties and Construction *by M. F. Kaplan*

Quality in Precast Concrete: Design — Production — Supervision *by John G. Richardson*

Reinforced and Prestressed Masonry *edited by Arnold W. Hendry*

Concrete Structures: Materials, Maintenance and Repair *by Denison Campbell-Allen and Harold Roper*

Assessment and Renovation of Concrete Structures *by E. Kay*

Concrete Bridges: Design and Construction

A. C. Liebenberg
Consulting Engineer

Advisory Editors
Professor F. K. Kong
Emeritus Professor R. H. Evans
Professor E. Cohen
Professor F. Roll

Longman Scientific & Technical
Copublished in the United States with
John Wiley & Sons, Inc., New York

Longman Scientific & Technical,
Longman Group UK Limited,
Longman House, Burnt Mill, Harlow,
Essex CM20 2JE, England
and Associated Companies throughout the world.

Copublished in the United States with
John Wiley & Sons, Inc., 605 Third Avenue, New York, NY 10158

First published 1992

British Library Cataloguing in Publication Data
A catalogue record for this book is available from the British Library

ISBN 0 582 04281 X

Library of Congress Cataloging-in-Publication Data
Liebenberg, A. C., 1926–
Concrete bridges: design and construction/A. C. Liebenberg.
p. cm. — (Concrete design and construction series)
Includes bibliographical references and index.
ISBN 0 470 21865 7 (Wiley)
1. Bridges, Concrete—Design and construction. 2. Reinforced concrete construction. I. Title. II. Series.
TG340.L54 1992
624'.257—dc20 92-5592
CIP

Set in 10/12 pt Times.
Printed and Bound in Great Britain
at the Bath Press, Avon

Contents

@Seismicisolation

Preface

This book is an extended version of a chapter on 'Bridges' by the same author in the *Handbook of Structural Concrete* which was edited by F.K. Kong, R.H. Evans, E. Cohen and F. Roll and published by Pitman Books Limited of London and McGraw-Hill of New York in 1983. In addition to sections not previously included, it also contains various improvements and updates of the older text. It is intended as a reference work on the current state of the art and science of design and construction of concrete bridges to meet the needs of practising civil and structural engineers as well as students. However, it is not intended to be a comprehensive treatise on concrete bridges but rather a complementary work to other publications and codes of practice that deal more systematically with the details of practical design and construction. More emphasis is given to subject matter and concepts that are not always covered in textbooks but nevertheless are very important to the understanding of the design, construction and maintenance of concrete bridges. Reference is made to other chapters of the *Handbook of Structural Concrete* as well as other publications covering matters that have been omitted due to restrictions on length.

As knowledge of the past is essential in any field of endeavour this work commences with an historical summary of the development of the art of bridge building towards the numerous types and configurations of modern concrete bridges. The importance of the appreciation of aesthetics and the protection of the natural environment is emphasized. The need for an understanding of the underlying philosophy of the structural design process, including the nature of innovation and the concepts of risk, reliability and utility, is explained. The underlying thesis is that the final design product should in theory be optimized in terms of total utility, which includes all the relevant costs, benefits and damages to the immediate client, society in general and the natural environment over the life span of the bridge.

Relevant design criteria, such as the various actions and environmental effects on bridges, are defined and a summary given of suitable analytical procedures based on a range of mathematical modelling techniques and computer applications. Design and construction practices are described and the importance of field inspections and maintenance procedures emphasized. The causes and nature of the deterioration of concrete bridges are enumerated as well as methods of assessment of the damages. Effective repair or rehabilitation techniques are explained.

Acknowledgements

The author wishes to acknowledge the assistance received from the following persons in preparing portions of the first drafts of the sections listed: Mr V. Trümpelmann (5.2, 5.6 and 5.7); Professor R.D. Kratz (4.2 to 4.6) and Mr W. Göhring (5.3 to 5.5, 5.8 and 5.10); and from Mr S. Roux for researching the references, Messrs M. Minnaar and M. Quarmby for preparing the line illustrations, his son Louis Liebenberg for the pen sketches in Chapter 1, and all those who supplied photographs as acknowledged. He further wishes to acknowledge the use of extracts from the contents of various papers, reports and textbooks by several authors as indicated in the text or by superscript numbers, which refer to publications listed under the references. Special mention must be made of almost five pages extracted from a paper by J.P.H. Frearson read at a symposium of the Concrete Society in London in 1985.[6.22] However, most references are intended for further reading of the relevant subject matter.

Notation

A_{ef}	effective torsion area
A_{eq}	equivalent shear area
A_s	area of reinforcing steel
A_{sh}	shear area
a, b	diagonal and off-diagonal position indicators of the independent bending stiffness terms of the member matrix respectively
B	expected present value of the overall benefits derived from the existence of the structure (positive utility)
$\mathbf{C}$	constant damping matrix
C_i	initial cost
C_m	capitalized normal maintenance costs
C_p	capitalized prime costs $= (C_i + C_m)$
C_t	total capitalized costs (negative utility)
d_{ef}	diameter of largest circle within the boundaries of A_{ef}
E	Young's modulus or modulus of elasticity
E_c	initial tangent modulus of concrete
E_d	expectation of damages
E_{dyn}	dynamic modulus of elasticity
E_H	standard hydrogen potential
E_{long}	long-term modulus of elasticity
E_S	Young's modulus of steel
E_{sec}	secant modulus of concrete
E_t	time-dependent modulus of elasticity
e	eccentricity
F	force on a member; safety factor
$\mathbf{F}$	internal member stress resultant vector
F_R	probability distribution function of the structural capacity, R
F_x	equivalent static earthquake load
f	foundation factors used in conjunction with earthquake loading

$\boldsymbol{f}$	member fixed-end vector
f_S	probability density function of the load effect, S
G	shear modulus
g	acceleration due to gravity
h_{ef}	equivalent wall thickness used to define torsion stiffness
I	importance factor of structures used in conjunction with earthquake loading; moment of inertia or second moment of area of a section (sectional inertia)
I_d	effective sectional inertia for an applied deflection
I_{eq}	equivalent sectional inertia
I_r	effective sectional inertia for an applied rotation
J	torsional moment of inertia
$\boldsymbol{K}$	stiffness matrix
k	member stiffness
$\boldsymbol{k}$	member stiffness matrix
k_d	deflection stiffness in bending
k_{gh}	peak horizontal ground acceleration
k_r	rotational stiffness in bending
k_t	torsional stiffness
L	length of a member; design life in years; loaded length of bridge deck
L_w	wind-loaded length
M	bending moment; safety margin
$\boldsymbol{M}$	constant mass matrix
m	mass
m_M	mean safety margin
m_R	mean resistance
m_S	mean applied force
N	judgement factor
n	number of degrees of freedom in a structure; number of piles in a group
n_0	fundamental natural frequency
P	load or force
$P\Delta$	the bending moment increment due to column sway or drift
$\boldsymbol{p}$	stress resultant
p	probability of an event
p_f	probability of failure
p_m	probability of exceeding a serviceability limit state
p_n	probability of reaching an ultimate limit state
p_s	probability of survival
R	redistribution factor; resistance; structural capacity; reliability
$\boldsymbol{R}$	external nodal point load vector

R_D	resistance of a section for design purposes
r	minimum number of piles required to ensure statical equilibrium
S	applied force; load effect
S_D	force on a section for design purposes
S_a	the seismic response factor for a structure read from the response spectrum
S_m	capitalized cost of damage or loss due to non-compliance with serviceability criteria
s	stirrup spacing
T	return period; natural period of vibration
$\boldsymbol{T}$	transformation matrix
t	time
U_n	capitalized cost of reaching an ultimate limit state
$\boldsymbol{U}$	time-dependent displacement vector
$\dot{\boldsymbol{U}}$	time-dependent velocity vector
$\ddot{\boldsymbol{U}}$	time-dependent acceleration vector
u_{ef}	polygonal perimeter length of section in torsion
V	coefficient of variation
v	mean hourly wind speed
W	load
β	safety or reliability index
Δ	an increment
Δ_c	compression
γ_i	modal participation factor used for earthquake loading
δ	deflection of a member or support
η	relaxation coefficient of concrete
θ	rotation of a joint
λ	fixity factor used to determine member stiffness
μ	ductility factor of the structure; coefficient of friction
Π	product of
Σ	sum of
σ	variance
ϕ or ϕ_t	coefficient of creep of concrete to be used for long-term load effects
Φ	normal distribution function
Ψ_{ij}	normalized mode component of the ith mode in the direction of the jth degree of freedom
ω	radial frequency

@Seismicisolation

1 Introduction

1.1 General

In the following sections an account is given of the historical development of the art and science of bridge building from the primitive beginnings to the magnificent structures of today. There are many useful lessons to be learnt, not only from the progress through time but especially from the many failures. The importance of the parallel developments in mathematical analysis and the technologies of construction materials and techniques is very evident. It is also clear that at any time the available construction materials played a dominating role in determining the types and ranges of feasible and viable structures. This situation still pertains today; thus there is a place for a book devoted to concrete as the construction material which, in conjunction with steel as reinforcing, dominates the scene in many countries. This does not ignore the fact that many, including the biggest, bridges are still being built of steel and always will be of some metal alloy or other. It simply recognizes the fact that concrete bridges, whether ordinarily reinforced, prestressed or composite, are in a class distinct from that of structural steel, with very different design approaches, and accordingly they have become a specialized field. Nevertheless, the designer in concrete must have a substantial understanding of structural steel design codes and experience in the design of steel and timber structures because of their important applications in the form of temporary construction supports, launching girders and towers.

The continuing improvements in the quality and range of concretes as well as reinforcing steel and steel used in prestressing or in composite construction furthermore open challenging opportunities for the future applications of concrete. The evolution of large-span cable-suspended concrete bridges presents exciting possibilities and makes them potential competitors of longspan steel bridges of up to 700 m (2300 ft) clear span. However, the engineering profession, in applying concrete to such a wide range of applications over the years, often erred in not giving sufficient attention to all the details of construction and especially to the need for continuing site inspections and maintenance of bridges which are essential to ensure a long life.

The general approach to design is based on ISO 2394, *General Principles for the Verification of the Safety of Structures*[1.1] as expounded in Volumes I and II of the *International System of Unified Standard Codes of Practice for Structures*[1.2] published by the Comité Euro-International du Béton (CEB) in 1978. This code has been revised and a First Draft of CEB–FIP Model Code 1990 — has been printed. As they are model codes, reference is also made to specific codes that are being applied in practice or are in the process of being introduced.

The units in this book are those of the SI (International System of Units), but, where it is useful, imperial dimensions are also given.

1.2 A Summary of the Historical Development of the Art and Science of Bridge Building

1.2.1 Ancient History

Bridges have fascinated mankind from the earliest recorded times. Apart from their civil and military importance, they are considered by many as symbolic of civil engineering achievement. Although the origins are uncertain, the evolution of the various types of bridge structures probably covers a period exceeding 5000 years. One can speculate with fair certainty that the bridge types that are described in the next section were foreshadowed in the earliest constructions to span rivers and gorges. These included the primitive use of timber logs or stone lintels as beams or slabs, stone clapper bridges, boulders in crude masonry arches and ropes made of creepers, vines or woven natural fibres in small suspension bridges. The subsequent development by empirical methods can clearly be related to the advancement of successive civilizations and their knowledge of materials of construction. Various authors have covered the major periods in the history of bridges in some detail[1.3–1.5] and made reference to source material dating back more than four millennia. The history of the development of bridges makes most interesting reading and an in-depth study of the above references is essential for any prospective bridge designer. Only a few salient developments are given here.

The earliest bridge on record was that built on the Nile by Menes, the first king of the Egyptians, about 2650 BC. No details of this bridge are known but a remarkable bridge, with a timber deck on stone piers as described by Diodorus Siculus, was built over the Euphrates in Babylon 4000 years ago. Primitive suspension bridges, where the traveller slid along a single cable made of strands of bamboo rope twisted together, were made in India. The first true suspension bridge consisted of three cables: two on each side to act as handrails to enable the passenger to walk on the third, which was tied below. The oldest extant chain bridge is thought to be that over the river Tchin-tchin in China. India is noted for its early use of iron suspension chain bridges. Another form of primitive bridge in timber was a girder type on floating supports, which was the forerunner of the pontoon bridges used in modern warfare.

The development of the brick or masonry arch can be traced back to the Chaldeans and Assyrians but was apparently developed independently in the Western world. Until the time of the Roman conquest of Persia, it appears that these mostly took the form of corbelled arches with a pointed profile built of brick or stone in horizontal courses. These have been found widely distributed over various parts of the ancient world including China, the Middle East and Mexico. The first arches with voussoirs were most probably built in Egypt, where tombs dating from 1800 BC were discovered with roofs of elliptical profile. The Persians built arches with ogival or pointed as well as elliptical profiles. The Etruscans, the immediate predecessors of the Romans, developed the semicircular arch built with voussoirs. There is a difference of opinion amongst historians as to whether the Romans absorbed this knowledge during the Roman Conquest or learned about the arch directly from Eastern people. The Greeks had developed an elementary theory of statics and Archimedes (287–212 BC) understood the basic conditions of equilibirum. There is, however, no evidence from ruins that they built true stone arches. Even the Romans apparently never had a full understanding of how arches acted to resist the forces generated by the applied loads. However, they have gone down in history as the greatest builders of stone-masonry arches, almost without exception of circular profile and of comparatively small span and heavy proportions. From experience, observation and deduction within the limits of their understanding, they perfected the art of construction to the extent that many examples of their arches, such as the masterpiece of Agrippa, the Pont du Gard, 19 BC (Fig. 1.1), are to this day visible evidence of their creative engineering ability.

The Greeks and Romans used timber for less permanent structures. Timber trestle bridges were developed for military purposes. They built the first timber bridge across the Thames in London.

Perhaps the most important discovery of the Roman builders was that of natural cement, pozzolana, a volcanic sand found at Pozzuoli near Naples, that forms a hydraulic cement when mixed with ordinary lime and was the forerunner of modern concrete.

Bridge builders in those ancient times were practical men who unfortunately did not readily avail themselves of the ideas of philosophers, who were interested in the early developments of applied science. This

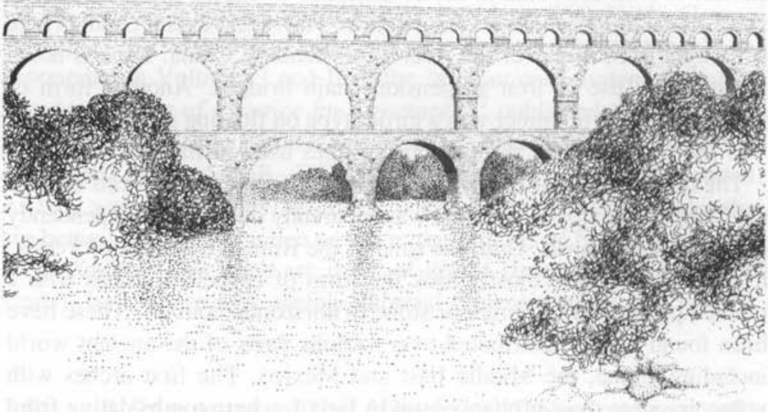


Figure 1.1 Pont du Gard — Agrippa's masterpiece, 19 BC (sketch based on photograph in Ref 1.6)

lack of communication delayed the proper understanding by bridge builders of the structures that they were developing by empirical methods, for many centuries.

1.2.2 The Middle Ages

A great deal of the knowledge of structural engineering accumulated by the Romans was lost during the Middle Ages, so that little progress was made and hardly any structural innovation took place. The art did, however, spread more widely throughout Europe. Chinese timber arch bridges built in the 10th century are still in use. How far the technical as distinct from empirical knowledge of bridge designers had developed by the end of this period is not known.

1.2.3 The Renaissance and the Age of Reason

The revival in the arts and sciences under the leadership of men such as Leonardo da Vinci (1452–1519) that occurred during the Renaissance had a fundamental influence on engineering and especially on bridge design. It heralded the birth of what can be considered to be the modern bridge in that it brought about a profound marriage between the conceptual understanding of structures based on theoretical considerations and the practice of bridge construction. This elevated bridge building to an applied science. Practical builders could no longer ignore the ideas of theoretical designers. Several of the great names associated with bridge building in the ensuing periods were to be men who combined both abilities.

The most notable bridges which were the earliest manifestation of this creative upsurge in the 16th century were the Santa Trinità Bridge in Florence, the picturesque Rialto Bridge spanning the Grand Canal in Venice and the Pont Notre Dame and Pont Neuf in Paris.

Using the ideas of men such as Leonardo da Vinci[1.6] and the results of their experimental work, it was now possible, for example, to determine the correct lines of thrust and the resultant forces acting in arches. The notion of the principle of virtual displacements made it possible to analyse various systems of pulleys and levers such as are used in hoisting devices. Da Vinci also studied the strength of materials such as iron wires by carrying out breaking tests by ingenious methods, and he carried out tests on beams and columns. Although he understood the method of moments, he did not adequately solve the problem of the internal resistance to bending of a structural member. It is probable that he invented the timber truss.

Galileo (in about AD 1600) made great strides by establishing various relationships between beam dimensions and their resistance to loads, and he came to the important conclusion:[1.6]

> You can plainly see the impossibility of increasing the size of structures to vast dimensions either in art or in nature: likewise the impossibility of building ships, palaces, or temples of enormous size in such way that their parts will hold together. This can only be done by employing a material which is harder and stronger than usual, or by enlarging the size or shape of the members. (From S.P. Timoshenko, *History of Strength of Materials*, © 1953, published by McGraw-Hill Book Company, New York, and reproduced with permission.)

He realized the advantages of hollow sections. He understood the concept of stress but did not solve the problem of the bending resistance of beams.

Academies of science originated in Italy in 1560 and the 17th century saw similar developments in various other countries, e.g. England, France and Germany.

The knowledge of what became known as the *strength of materials*[1.6] advanced significantly in the 17th century with important contributions by Robert Hooke (1635–1703), whose major discovery was the law named after him; Mariotte (1620–1684) for his studies in bending theory; Jacob (1654–1705) and John Bernoulli (1667–1748), the two famous brothers who produced theories for the deflection of beams, based on the differential and integral calculus developed separately by Newton (1642–1727) and Leibnitz (1646–1716), and also formulated the principle of virtual displacements; Daniel Bernoulli (1700–1782), the son of John Bernoulli, for his investigations into the shapes of curves which a slender elastic bar will take up under various loading conditions and for deriving the differential equations governing lateral vibrations of prismatic bars; and Leonard Euler (1707–1783), who extended Daniel Bernoulli's work and furthermore derived the famous formula for the

@Seismicisolation

critical buckling load of compression members. He also explored the theory of deflection and vibration of membranes.

During the 18th century, the scientific results of the preceding 100 years found practical applications and scientific methods were gradually introduced in various fields of engineering. There was a rapid spread of knowledge through the publication of various books and the calculus became a useful tool to many practising engineers.

In 1713, Parent (1666–1716) published two very important memoirs in which he for the first time clarified the importance of the position of the neutral axis and correctly solved the internal stress distribution in a beam.

The first engineering school in the world was formed by the Corps des Ingénieurs des Ponts et Chaussées in Paris in 1716 and at that time the first books on structural engineering were published. The influence of these movements could soon be noticed in the evolution of new designs such as Perronet's design of the Neuilly Bridge over the Seine, which had piers with a thickness of less than one-ninth the span of the arches.

The longest span truss bridge ever built in timber was the Wittengen Bridge over the River Limmat in Baden, Germany, in 1758. It was destroyed by the French army in 1800.[1.7]

C.A. Coulomb (1736–1806) played a very important role in this period, covering a wide field. His main contributions to structural engineering were his experimental and theoretical work on the bending of beams and torsion of rods, the stability of retaining walls and the theory of arches. At this time, experimental testing to obtain the mechanical properties of structural materials became well established.

1.2.4 The Industrial Revolution

The later decades of the 18th century saw the first application of the products of the Industrial Revolution to structural engineering in England. The first cast-iron bridge to be built in the world, the famous Coalbrookdale Bridge (Fig. 1.2), a semicircular arch with a span of 30.5 m (100 ft) over the River Severn, built by Abraham Darby and probably based on designs by Thomas Pritchard and John Wilkinson, was completed in 1779.[1.8] The first iron bridge in America was built 60 years later.

In the early part of the 19th century, very significant advances were made in the theory of the mechanics of materials by Navier (1785–1836), but it took several decades before engineers began to understand them satisfactorily and to use them in practical applications. This work heralded a new period in engineering and was probably the beginning of modern structural analysis. Navier was the first to evolve a general method of analysing statically indeterminate problems. His work was followed by

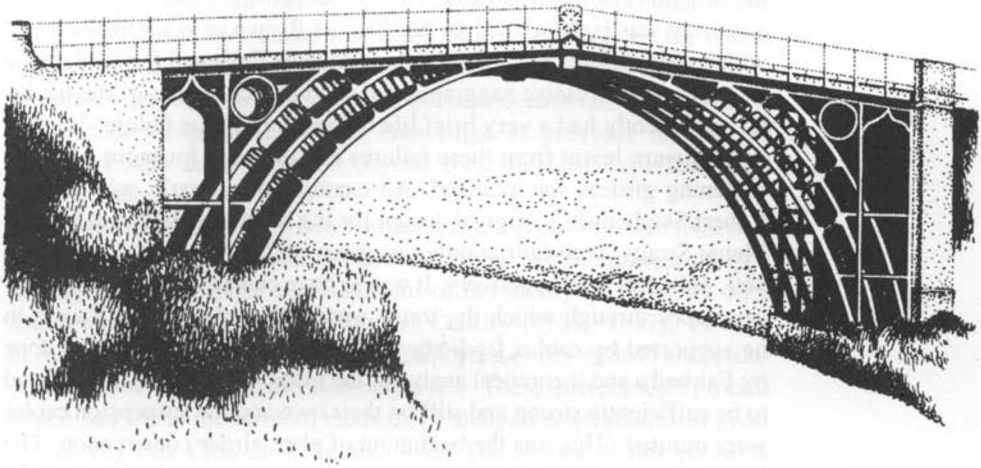


Figure 1.2
Coalbrookdale Bridge

the major contributions of other famous mathematicians, the scientists and engineers whose works have been well documented[1.6] and form the basis of modern structural engineering.

1.2.5 Early Iron and Steel Bridges

The 19th century saw the rise of the great iron and steel bridges, in which Great Britain took the lead, and included a large variety of configurations. Initially, the proposals were mainly in cast iron and some very bold designs were proposed by Watt, Telford, the Stephensons, Brunel and others. A cast-iron arch of 183 m (600 ft) span, proposed by Telford, was never built because of various objections and it was many years before an arch of comparable span was built in steel. After doing tests on the tensile strength of malleable iron in 1814, Telford developed a design for a suspension bridge with a main span 305 m (1000 ft) long and two side spans of 152 m (500 ft). He retained this idea for the design of the now famous 177 m (580 ft)-span suspension bridge over the Menai Straits, completed in 1826. Several major suspension bridges were built in this period but, unlike Menai (re-chained and strengthened in 1938–1939), few survive today.

The earlier suspension bridges were designed to carry only horse-drawn roadway traffic and some of these were inadequately designed to resist wind forces and the dynamic effects of traffic and were consequently subjected to disquieting movements. Several failures occurred, including that of the Broughton chain bridge at Manchester in 1831 which was due to the oscillations set up by a troop of soldiers marching in step to a drum and fife. Various other bridges had failed in windstorms in England, Germany, France and America. A notable case was that of the Brighton chain pier in 1833. The first suspension bridge which carried a railway was built in 1835 by Sir Samuel Brown over the River Tees, with a span of 85.6 m (281 ft). It was actually designed for road traffic and suffered excessive sagging under the heavier weight of the trains. It consequently had a very brief life because of fatigue failure. Various lessons were learnt from these failures and the need for more adequate stiffening girders was realized. An example of an early solution was Robert Stephenson's original design for the Britannia Bridge across the Menai Straits, with four continuous spans of 70, 140, 140, 70 m (230, 460, 460, 230 ft) respectively. It was to have consisted of two wrought-iron tubes through which the trains were to travel and which were to be supported by cables from above. On the basis of model tests done by Fairbairn and theoretical analyses, the tubes were subsequently found to be sufficiently strong and stiff on their own and the suspension cables were omitted. This was the beginning of plate girder construction. The bridge was opened to traffic in 1850. It was damaged by fire in 1970 and reconstructed with steel arches replacing the tubes.[1.7] I.K. Brunel[1.9] designed the Royal Albert Bridge at Saltash, completed in 1859, in which he combined the principles of arch and suspension bridge to form the two 139 m (455 ft) main spans. For the arch he used a massive wrought-iron tube. He also designed the Clifton suspension bridge with a span of 214 m (702 ft), completed in 1864 after his death. The 122 m (400 ft) span Albert Bridge over the Thames (1873) combined a catenary suspension system with stays.

The advance of railways had given bridge building a major impetus, so much so that more than 25 000 bridges were built in Great Britain in 70 years. The heavier and more concentrated railway live loading, with its hammer blow and other dynamic effects, required more rigid structures and arches, and cantilever trusses replaced suspension bridges in the medium to long-span range. In the USA, a large number of masonry arches and patented trusses, mainly of composite wood and wrought iron,[1.10] were built for the rapidly expanding railroad system.

In the second half of the 19th century, steel superseded iron in the superstructures of bridges although it took time before it was generally accepted with confidence. It introduced the era of large steel bridges. One of the famous early bridges to be built in steel was the St Louis Bridge over the Mississippi River, designed by J.B. Eads and completed in 1873.[1.11] This was the first big bridge of steel arches to be erected by the modern cantilever method. It consisted of three arches with a central span of 158 m (520 ft).

In 1879 the tragic collapse of the Tay Bridge occurred. This badly shook the confidence of engineers and led to a greater appreciation of the magnitude of wind forces. The disaster was caused by lack of aeroelastic stability, whereas the early suspension bridge failures were blown down because (as is now realized) they had insufficient aerodynamic rigidity. There was insufficient knowledge of the nature and magnitude of wind forces and the response of structures. The history of failures demonstrates the courage, perhaps occasional foolhardiness, and almost inexplicable lack of knowledge of previous failures, of engineers who have ventured beyond their field of experience and understanding.

Significant advances had however been made in construction materials and methods of design, and the understanding of structures developed rapidly due to the prior establishment of the theory of elasticity on a sound basis by men such as Navier. Cauchy (1789–1857), Poisson (1781–1840), Lamé (1795–1870), Clapeyron (1799–1864) and Saint-Venant (1797–1886), to mention only a few. These men started a process which had led to the modern methods of analysis of structures of great complexity (see Chapter 4, Section 4.1). The history of this period is particularly interesting and has been well researched by others.[1.6] Experimental work had also become a well-established discipline due to the pioneer work of William Fairbairn (1789–1874), Eaton Hodgkinson, Weisbach and others. The progress that was made with major contributions by Clerk-Maxwell (1831–1879) and Karl Culman (1821–1881) was such that Sir Benjamin Baker could by 1880 design his masterpiece, the Forth Bridge, by calculation alone. He had to resort to experiments, however, to determine the magnitude of the wind forces.

A large range of interesting steel bridges was built in Europe in this and the ensuing period, including movable bridges (bascules, swing spans, and the vertical lift types). With the development of the New World and the British Empire, the construction of roads and railways required a large number of bridges. In the USA and Canada, Australia and New Zealand, many of the early bridges were built in timber. In Southern Africa, the early bridges were mainly of the through type with steel trusses fabricated in Great Britain and transported in sections by sea and ox wagon to the sites. Several of these bridges built towards the end of the 19th century still survive although they are rapidly being replaced by modern bridges. Several notable steel bridges were built in South America, Africa and India.

The Forth Bridge, which when completed in 1887 introduced the era of long-span cantilever truss bridges, was designed by Sir Benjamin Baker in partnership with Sir John Fowler. It consisted of two main spans of 521 m (1710 ft) made up of two 207 m (680 ft) cantilever arms and a 125 m (350 ft) suspended span, all of truss configuration with the main compressive members being tubular steel struts.

This was followed in 1909 by the Queensboro Bridge over the East River in New York City, with a main span of 360 m (1182 ft), and in 1918 by the Quebec Railway Bridge over the St Lawrence River in

Canada, with a main span of 549 m (1800 ft), which at that time was the longest span in the world. The tragic collapse[1.3] of the first design in 1907 and the collapse of one suspended span of the revised design during construction in 1916 were further examples in the long line of collapses which were to be repeated several times more in spite of the lessons learnt. Another notable steel cantilever bridge, opened in 1943, is the Howrah Bridge over the Hooghly River in Calcutta, which has a single main span of 457 m (1500 ft), with two short anchor arms.[1.4]

Following on the success of the St Louis arch bridge in 1873, a large number of large steel arches have been built. Most famous are the 152 m (500 ft)-span Victoria Falls spandrel-braced arch over the Zambezi River, designed by Sir Ralph Freeman, Snr (1880–1950), under the direction of G.A. Hobson; the 298 m (977 ft) span Hell Gate Bridge over the East River in New York, designed by Gustav Lindenthal and O.H. Ammann and completed in 1916; the 504 m (1652 ft) span Bayonne Bridge across the Kill van Kull at New York, designed by Ammann and completed in 1931; the Sydney Harbour Bridge completed in 1932 with a span of 503 m (1650 ft), which had been conceived by Dr J.J.C. Bradfield after studying the Hell Gate Bridge and was designed for the contractor by Sir Ralph Freeman (Snr), who also designed the 329 m (1080 ft) span Birchenough arch bridge over the Sabi River in Zimbabwe, which was completed in 1935. This latter bridge has very slender and beautiful proportions. It was probably the first instance of the use of wind-tunnel tests on scale models to establish the wind forces on a structure. In the same period, Freeman designed the 320 m (1050 ft) span Otto Beit suspension bridge over the Zambezi at Chirundu (completed in 1939). This is another example of a slender structure built at minimal cost and in less than two years in a relatively undeveloped region. It was also the first suspension bridge of over 1000 ft span to be designed and built outside North America, and its parallel-wire cables were formed by a unique method.

1.2.6 Prewar Long-Span Steel Suspension Bridges

In the USA, the art of bridge building had made significant advances under the leadership of several able designers and builders, amongst whom John A. Roebling (1806–1869) can be considered the inventor of long-span suspension bridges. He designed the famous Brooklyn Bridge (1883), which, at 486 m (1595 ft), was the first long-span bridge to be built of steel and in which he used inclined stays as well as the vertical hangers to support the superstructure. Because of his untimely death, it was completed by his son Washington in 1883. The success of this bridge was largely responsible for a renewed confidence in suspension bridges and it became the forerunner of many of the great suspension bridges in America, such as the George Washington Bridge with a main span of 1067 m (3500 ft) over the Hudson River, designed by O.H. Ammann and completed in 1931. The famous Golden Gate Bridge with a main span of 1280 m (4200 ft), was completed in 1937. The Verrazano Narrows Bridge, with a main span only 18 m (60 ft) greater, was completed 28 years later.

1.2.7 The Birth of Reinforced Concrete

It was towards the end of the 19th century that reinforced concrete was for the first time significantly applied to bridge construction (see also Chapter 1, Section 2; Chapter 2, Section 2; and Chapter 17, Section 1.2 of Ref. 1.12). Although crude attempts at reinforcing various forms of concrete are evident in the ruins of ancient Rome, these ideas lay dormant for 18 centuries until Smeaton's experiments in 1750.[1.13] Louis Joseph Vicat (1786–1861) carried out research on cement mortars prior to 1818 and Joseph Aspdin (1779–1855) patented Portland cement in 1824.[1.3] J.L. Lambot built a rowing boat of concrete reinforced with a rectangular mesh of iron rods in 1848, and Francois Coignat constructed a concrete building in 1852 by casing an iron skeleton framework in concrete.[1.13] The real innovator, however, was Thaddeus Hyat (1816–1901), who carried out experiments in the USA in the 1850s with flat wrought-iron bars on edge which he bonded to the concrete with transverse rods inserted through holes therein.[1.3] He published a book in 1877 on his experimental work. He recognized the importance of the near-equality of the coefficients of thermal expansion of concrete and iron and assumed a modular ratio of 20.[1.13]

The concept of a composite structural material, combining the high crushing strength of concrete with the high tensile strength of iron, was, however, largely developed in France around the turn of the century by François Hennebique, Edmund Coignet, N. de Tedesco and Armand Considère. The real credit for laying the foundation of the reinforced concrete industry in about 1867 seems to belong to Joseph Monier. W.B. Wilkinson had patented a reinforced concrete floor system in England in 1854, but his ideas were not widely adopted.[1.3,1.13]

An early example of a reinforced concrete beam bridge is Homersfield Bridge in Suffolk, with a 15 m (50 ft) span, completed in 1870.[1.7] It was Wayss of Germany who made a first attempt at a design theory. Together with Freitag, he designed and constructed some 320 arched reinforced concrete bridges between the years 1887 and 1891 with spans of up to 40 m. M. Koenen was probably the first to publish, in 1886, an analysis of the behaviour of a reinforced concrete beam. A period of rapid development of various reinforced concrete systems followed in Europe and the USA. Hennibique designed and built the first notable reinforced concrete arch, the Pont de Chatellerault,[1.4] in 1898 with spans of 40, 50 and 40 m and a rise-to-span ratio of approximately one-tenth. The depth of the arch at the crown is only about one-hundredth of the span. His Risorgimento Bridge in Rome (1913), with a span of 100 m (305 ft), is considered to be a masterpiece for that period.

As with other structures, reinforced concrete, however, also experienced several failures in the pioneering days.

1.2.8 Pre-war Reinforced and Prestressed Concrete

Whereas steel bridges dominated in the long-span range, reinforced concrete gradually made progress in the short-span ranges, so that by 1900 Hennebique alone had been responsible for the design and construction of at least 100 concrete bridges. Melan introduced the system in which self-supporting reinforcement, mainly in the form of rolled-steel sections, acted as temporary support to the shuttering and wet concrete. In Germany, the firm of Wayss & Freitag published the first authoritative textbook, *Der Eisenbeton*, written by Professor E. Mörsch and dealing with fundamental aspects of reinforced concrete design. By 1897, such agreement of fundamentals had been reached that it was possible for Charles Robert to give the students of the Ecole des Ponts et Chaussées a course of instruction on the design of reinforced concrete.[1.13] Standard specifications for reinforced concrete were introduced by the Swiss Institute of Engineers and Architects in 1903, to be followed by various other European countries.

Robert Maillart (1872–1940) of Switzerland made a considerable

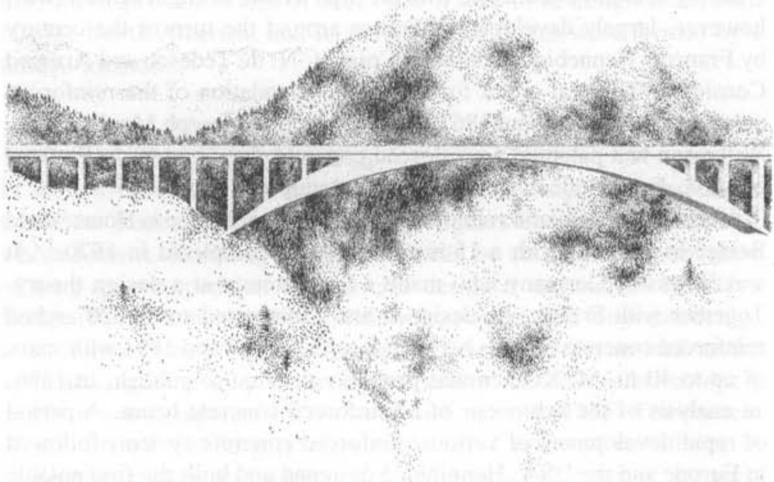


Figure 1.3 Maillart's Salgina Bridge in Switzerland, completed in 1930. Its span is 90 m (sketch based on photograph in Ref 1.5)

impact with the bridges he designed (Figs 1.3 and 1.4) and, with François Hennebique of France, played a leading role in establishing reinforced concrete in bridge building. Although largely an intuitive designer who did only a minimum of calculations, Maillart's mastery of both the technical and aesthetic aspects of design have left an indelible mark on the engineering and architectural professions.[1.3,1.4,1.14–1.16]

The early decades of this century saw the general acceptance of reinforced concrete and many short- and medium-span bridges were being built in this composite of materials.[1.17]

The *Autobahn* developed in pre-war Germany, and the *autostrada* in Italy, were good examples of the benefits of reinforced concrete applied to freeway bridge structures. Various reinforced concrete arches, varying in span from 145 m (475 ft) to 192 m (630 ft), were built in Europe before 1943, when the famous Sandö arch, designed by S. Höggbom, was completed over the Ångerman River in Sweden with a span of 264 m (866 ft), a thickness-to-span ratio of 1:100 and a rise-to-span ratio of 0.151. It was successfully completed after the initial centering, which consisted of a timber-framed tied arch, had failed, and was replaced by a timber trestle supported on piles. This bridge was a very significant achievement at that time in terms of its span, low rise, slender dimensions and the high strength of concrete. It was only exceeded in span 20 years later by the Arrábida arch in Portugal.

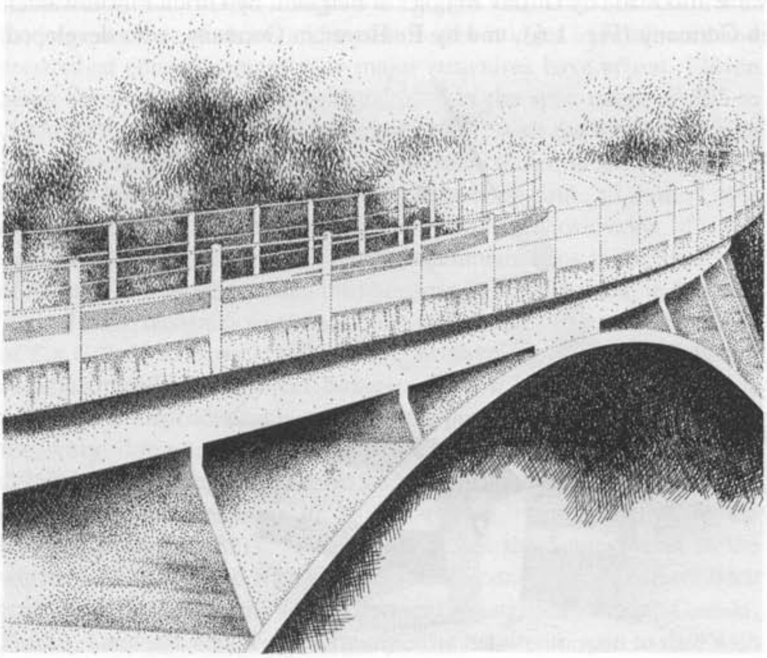


Figure 1.4 Maillart's Schwandbach Bridge in Switzerland, completed in 1933. Its span is 37.5 m (sketch based on photograph in Ref 1.43)

1.2.9 The Birth of Prestressed Concrete

Although the idea of prestressing masonry or concrete probably dates far back, its development only became feasible when high-tensile steels became available (see also Chapter 19, Section 1 of Ref. 1.12). Eugene Freyssinet (1879–1962) can be considered to be the father of prestressing[1.18,1.19] in that he played a leading role in inventing and developing one of the first effective systems which was applied to a wide range of concrete structures. He first used the method in 1907 in the construction of a tie joining the abutments of a test arch of 50 m span, but it took many years before prestressing was accepted by the profession. In fact, it was only in 1934 when he saved the new Ocean Terminal at Le Havre from collapse due to settlement, by post-tensioning applications, that the authorities became convinced of its uses. Being a practically minded man, he was closely involved in the construction of some of his bridges, whereby he gained a clear understanding of relevant problems such as that of deferred elastic strains and the loss of stress incurred due to various factors. His bridges, especially those over the Marne, were as epoch-making as those of Maillart, whose intuitive and practical approach he shared. His bridge at Orly (Fig. 1.5), with a main span of 53.3 m (175 ft), completed in 1959, is a masterpiece in composition of piers and deck.

Major contributions to the early theory and practice of prestressing were also made by Gustav Magnel in Belgium, by Ulrich Finsterwalder in Germany (Fig. 1.6), and by E. Hoyer in Germany, who developed

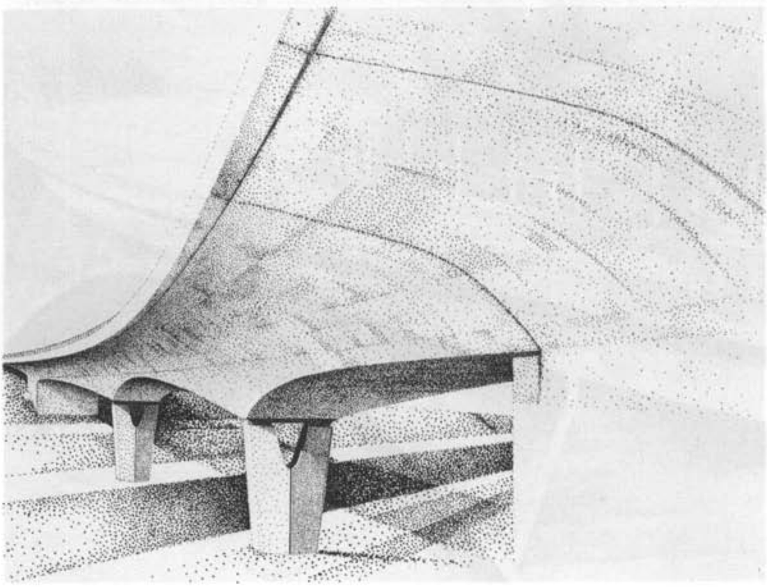


Figure 1.5 Freyssinet's bridge at Orly, completed in 1959. Its main span is 53.3 m (sketch based on photograph in Ref 1.3)

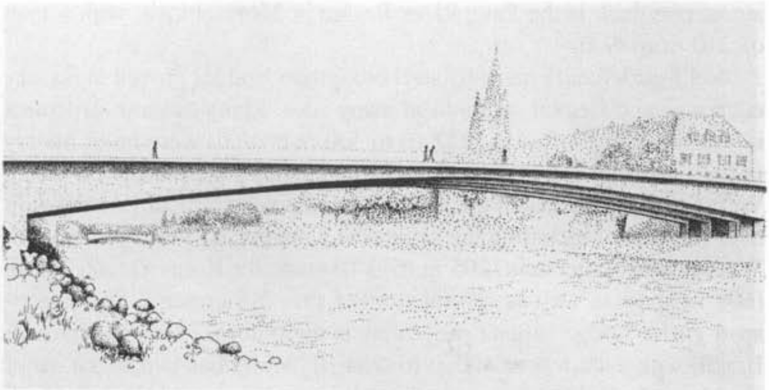


Figure 1.6
Gänstor-Brücke at Ulm, completed in 1950 (sketch based on photograph in Ref 1.5)

the long-line pretensioning method for producing factory-made prestressed concrete.

1.2.10 The Post-war Period and Modern Times

After the Second World War, as the economies of countries were revived, a resurgence in the building of a great diversity of bridges occurred in Europe to replace the large number destroyed. The development of the modern motorways generated a great demand in many countries for grade separation structures and bridges over gorges and rivers. In addition to the large number of short- and medium-span bridges in reinforced and prestressed concrete, many new major structures have arisen. Eleven major steel cantilever truss bridges[1.7,1.20] in the span range of 305 m (1000 ft) to 510 m (1673 ft) have been built, with the longest span in Japan and the others in the USA. Nine steel arches in the span range of 305 m (1000 ft) to 518 m (1700 ft) have been built, all in the USA, except one each in England, Canada and Czechoslovakia.[1.21] The world's only bridge of any size built of aluminium alloy is the fixed arch of 88 m (290 ft) span crossing the Saguenay River at Arvida, in Canada.

The British designers Freeman, Fox and Partners, under the leadership of Sir Gilbert Roberts (1899–1979), evolved an extremely light but aerodynamically stable steel box-girder deck in conjunction with a triangulated suspender cable system in the process of developing designs for very long-span suspension bridges in the 900–1500 m (2953–4921 ft) range. This form was first applied to the Severn (1966) and then to the Bosporus in Turkey (1973),[1.22] culminating in the Humber Bridge (1981)[1.23] which to date has the longest span in the world — 1410 m (4626 ft). Other notable suspension bridges have been built since the war in the USA, Portugal, Venezuela, Japan, Canada, France, Denmark and West Germany. The maximum span to date with a concrete deck is the Save River Bridge in Mozambique, with a span of 210 m (689 ft).[1.7]

Steel I-girder and especially steel box-girder bridges proved to be very economic and elegant solutions at many sites. Many of these structures in the span range of 80 m (252 ft) to 300 m (984 ft) were built, mainly in the European countries with well-established steel industries, where West Germany has taken the lead. Some imposing structures were built over the Rhine, including the bridges at Koblenz, 235 m (771 ft) span; Wiesbaden–Schierstein, 205 m (673 ft) span; the Köln–Deutz, 185 m (607 ft) span, as well as several at other sites in Germany. The largest-span girder bridge to date, however, is the Niteroi in Rio de Janeiro, Brazil, with a main span of 300 m (984 ft). More than two dozen major steel box-girder bridges (excluding cable-stayed ones) have been built in various countries since the war. The improvement in the quality of structural steels generally and the development of low-alloy steels with higher yield stresses increased the competitiveness of these steel structures. In Britain, a superior high yield stress steel (BS 968:1962) was produced and in the USA the weldable T1 steel had an even higher yield stress of 40 ton/in^2 (618 MPa). Welding both in the shop and at site replaced riveting, but the use of high-strength friction-grip bolts was developed for making connections in the field where welding facilities were not available or convenient.

Composite steel I- or box-girders with cast *in situ* or precast reinforced concrete deck slabs, proved to be very competitive in many cases (see also Chapter 17, Section 1.3 of Ref. 1.12).

This period also saw the revival of the cable-stayed girder bridge,[1.24] which in concept dates back to 17th-century Venice but is generally credited to Löscher (1784) in the form of a completely timber bridge. Redpath and Brown in England and the Frenchman Poyet, early in the 19th century, designed bridges with steel-wire cable- and steel bar-stays respectively. The first concrete structure to utilize cable-stays was the Tempul aqueduct with a main span of 60 m (198 ft), crossing the Guadalete River in Spain, designed by Professor Torroja and completed in 1925.[1.25] However, the first modern cable-stayed bridge with a steel deck, designed by F. Dischinger, a German engineer, was built at Strömsund in Sweden in 1955 with a main span of 183 m (599 ft) and a fan-type cable configuration supported on twin-column bents. In spite of initial scepticism, more and more cable-stayed bridges have been and are being built at sites suitable for this configuration.

The Strömsund was followed by the Theodor Heuss (North) Bridge over the Rhine at Düsseldorf in 1958, also with a steel box-girder deck and a main span of 260 m (853 ft), with side spans of 108 m (354 ft) and a fan-type cable configuration. The Severins Bridge at Cologne (1959) has a main span of 302 m (990 ft). The Lake Maracaibo Bridge

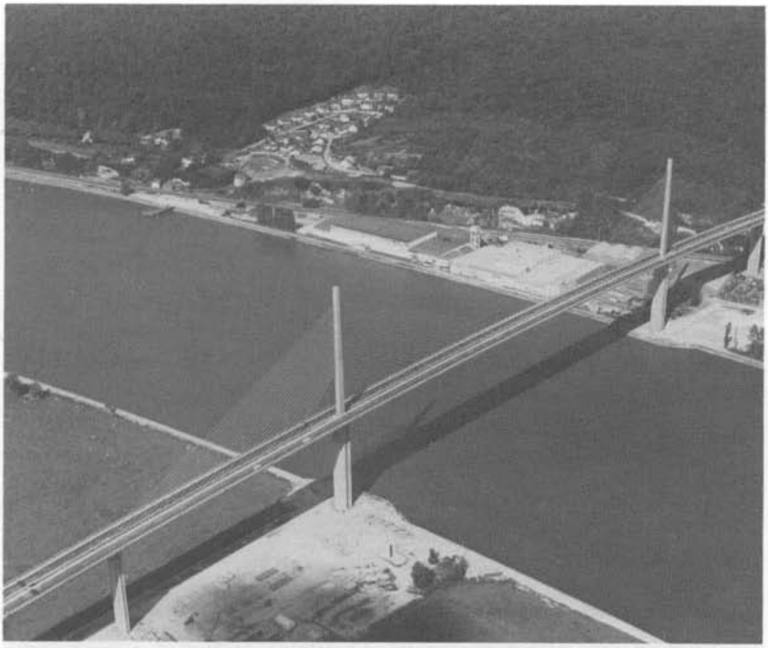


Figure 1.7 Brotonne Bridge over the Seine, completed in 1977. Its main span is 320 m (courtesy of Freyssinet International)

with spans of 235 m (771 ft), designed by Professor R. Morandi and constructed in 1962 (Fig. 1.22), is generally considered to be the first modern concrete cable-stayed bridge.[1.25] These bridges were followed by a large number with both steel and concrete deck constructions. The longest span to date with a steel deck is that of the Geislingen Bridge near Nuremberg in West Germany, with a main span of 651 m (2137 ft) and a fan-type cable configuration. The longest span to date with a concrete deck is that of the 1300 m (3650 ft) Brotonne Bridge (1977) over the Seine between Rouen and Le Havre (Fig. 1.7), with a main span of 320 m (1050 ft),[1.26] followed by Professor Morandi's 282 m (925 ft) span Wadi-Kuff (1971) at Beida, Libya, in which single sets of cables were used (Fig. 1.8).

A competition design submitted for the Danish Great Belt Bridge had a clear span of 345 m (1131 ft),[1.24] but was not awarded the first prize. A very novel proposal is that of the Ruck-A-Chucky Bridge in California,[1.27] a horizontal arc-shaped bridge in plan that will be suspended by cables anchored to the steep banks of the canyon (Fig. 1.9). The cable-stays fan out two-dimensionally so as to provide the necessary support for the curved concrete deck. The proposed span is 384 m (1260 ft) measured on the straight. The deck has a radius in plan of 457 m (1500 ft).

The most significant development in the post-war years has been the growth in the application of reinforced and prestressed concrete to bridge

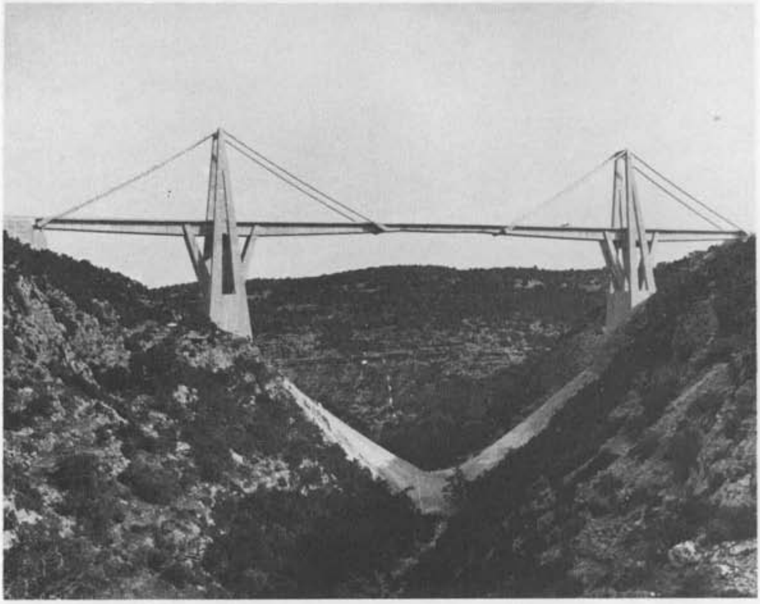


Figure 1.8 Professor Morandi's cable-stayed bridge at Wadi-Kuff, Libya. Completed in 1971, it has a main span of 282 m (courtesy of Professor Ing. Riccardo Morandi)

construction (see also Chapter 19, Section 1 of Ref. 1.12). A large number of prestressed concrete girder bridges have been constructed with various cross-sections including precast and composite construction as well as in-situ cast box-girders.

The free-cantilever system of prestressed concrete bridge construction was first applied in 1950 to a bridge across the Lahn River at Boldwinstein in Germany. Two years later this method was used to construct the 114 m (375 ft) main span bridge over the River Rhine at Worms[1.28] (Fig. 1.32). Since then, more than 100 bridges have been constructed in this way, including prestressed concrete box-girder bridges such as the Medway Bridge in England with a main span of 152 m (500 ft) (Fig. 1.10), and the bridge spanning the Rhine at Bendorf near Koblenz (Fig. 1.31), with a central span of 208 m (682 ft), which was the longest of its type in the world at that time (1963). Other notable structures of this type are the Urato Bridge at Shikoku in Japan (1972) with a main span of 230 m (754 ft), the Hamana–Chasi Bridge in Japan, 240 m (787 ft), and the Koror–Babelthuap balanced cantilever bridge, 241 m (790 ft), in the Palau Islands in the Pacific, completed in 1977. The first major prestressed concrete bridge in the USA, built in 1950, was the 49 m (160 ft) centre span Walnut Lane Bridge in Philadelphia, designed by G. Magnel, the Belgian pioneer of prestressed concrete. This resulted in prestressed concrete being well accepted in the States in spite of initial scepticism[1.29] and many major bridges have since been constructed by American engineers.

Figure 1.9 Ruck-A-Chucky Bridge in California (courtesy of Professor T Y Lin)

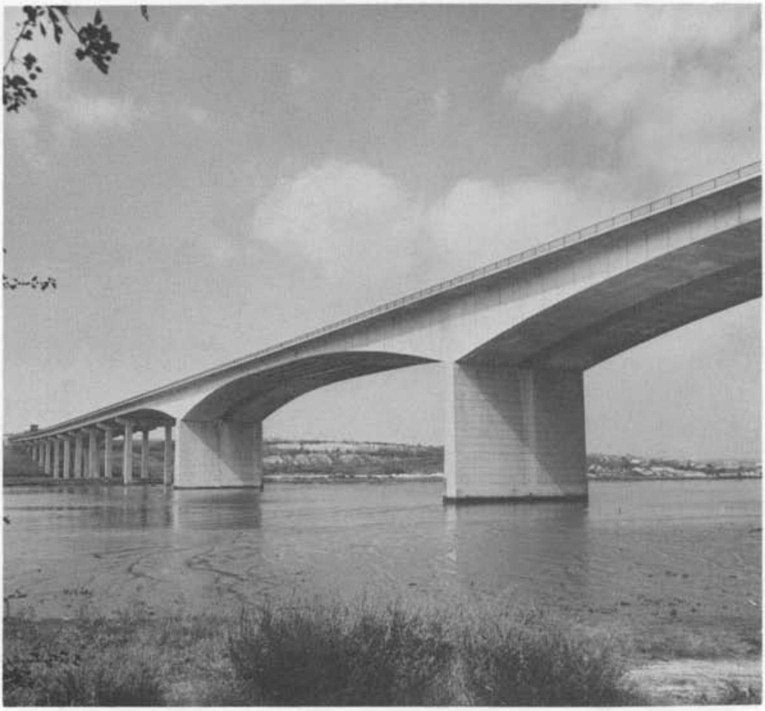


Figure 1.10 Medway Bridge, England, with a main span of 152 m (courtesy of Freeman, Fox and Partners)

The decks of many cable-stayed girder bridges are of reinforced and prestressed concrete construction. A significant feature of the construction of these bridges has been the diversity of construction methods that have been developed to increase their competitiveness. The various types of concrete girder and cable-stayed bridge are described, with reference to modern examples and methods of construction, in Chapter 5. The methods of analysis used in design are described in Chapter 4.

Since the Sandö Bridge, five reinforced concrete arches of larger span have been built. The world's longest-span concrete arch, the mainland to Island-of-Krk Bridge near Zagreb, Yugoslavia,[1.30] with a span of 390 m (1280 ft), was built by a cantilever method using temporary composite trusses anchored to the banks. The trusses were progressively constructed from opposite ends and incorporated the arch ribs and spandrel columns in conjunction with temporary upper chords and diagonals consisting of steel members and post-tensioned steel tendons. The Bloukrans Bridge in South Africa,[1.31] with a span of 272 m (892 ft) (Fig. 1.11), has been built by the suspended cantilever method using temporary suspension and tie-back (anchor) cables supported on temporary towers and anchored into the rock banks. This method of construction, which was first applied on a smaller scale on two arch bridges in Wales and on a larger scale on the 198 m (650 ft) Van Stadens

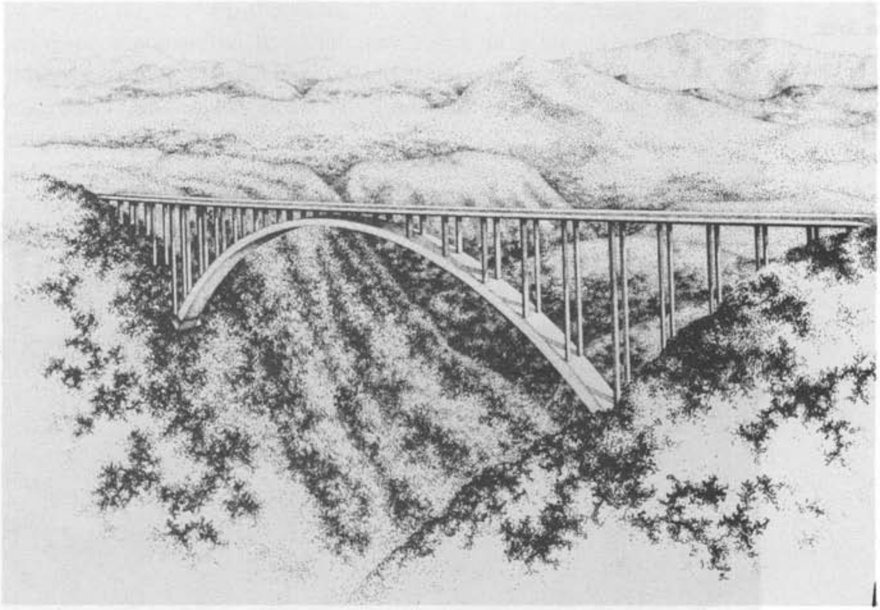


Figure 1.11
Bloukrans Bridge, South Africa, which was completed in 1983. The span of the arch is 272 m

gorge arch bridge in South Africa, is described in greater detail in Chapter 5. It has made the long-span arch competitive again after the more costly methods of construction with centering used on the 270 m (886 ft) span Arrábida arch at Porto, Portugal, in 1963, on the 290 m (951 ft) span Amizade arch over the Paraná-joki, Brasilia–Paraguay, in 1964, and on the 305 m (1000 ft) span Gladesville arch in Australia, in 1964 (Fig. 1.12).[1.7] The construction of the arch on the route between Caracas and La Guaira, in Venezuela, on articulated formwork supported by cables from towers on the abutments, can be considered a compromise between the two methods. In the late 1960s, prominent engineers had declared the supremacy of the cable-stayed girder bridges in the span range of 150–400 m, but the elimination of the need for expensive centering for arches has changed the picture considerably.

The longest span prestressed concrete truss to date is the Mangfall Bridge, in West Germany, with a main span of 108 m (354 ft), completed in 1960.

The long history of failures extending into modern times and including long-span bridges such as the 853 m (2800 ft) span Tacoma Narrows suspension bridge in the USA, which failed so dramatically in 1940 due to aerodynamic instability,[1.24] and various steel box-girder cable-stayed and other bridges which collapsed during construction in more recent times,[1.32] has taught the bridge-building fraternity expensive but invaluable lessons and led to greater efforts in the form of both experimental and theoretical work to understand the forces to which

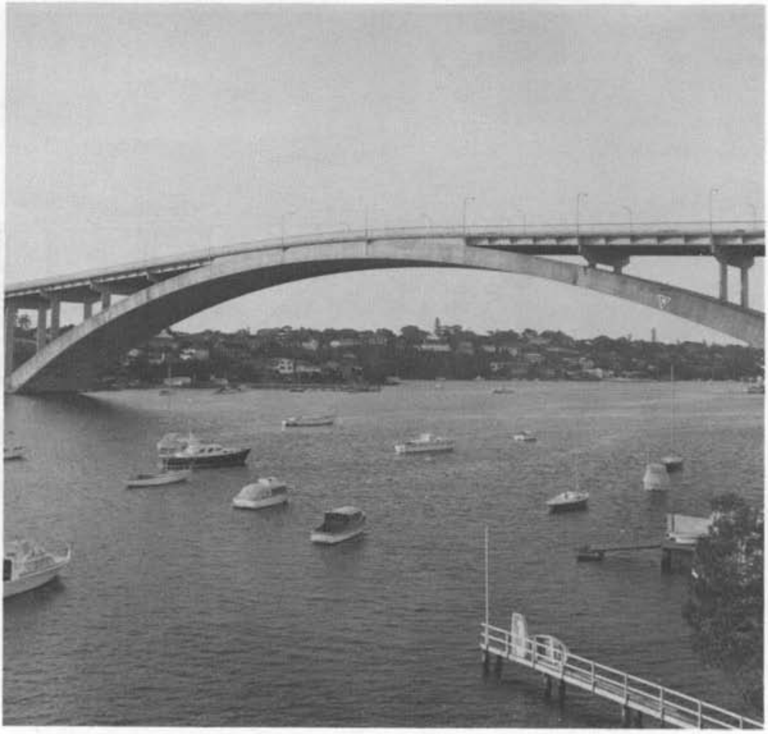


Figure 1.12 Gladesville Arch, Australia, completed in 1964. The span of the arch is 304.8 m (courtesy of Maunsell and Partners)

bridges are subjected and their response thereto.[1.33–1.36] This also applies to the effects of rivers in flood which have in many cases been underestimated leading to costly failures (see Chapter 2, Section 2.4.5). In time, all this experience has culminated in a new and sounder philosophical approach to design and construction practice, as discussed in Chapter 3.

Throughout its history, the availability of specific materials of construction has played a major role in determining the course of development of structural engineering. This is true today and the tendency for structural engineers to specialize in the use of one or other of the specific materials, e.g. structural steel rather than reinforced and/or prestressed concrete, or vice versa, has historical grounds in the development of virtually separate industries with different technologies. This specialization has been further reinforced by the sponsorship provided by the manufacturing or constructional concerns, of societies, institutes and publications that propagate the use of their particular material products and provide a very good information service as well as donations and bursaries for the education and training of engineers and technicians. These activities are praiseworthy in spite of being motivated by vested interests, as knowledge and competition are generally increased thereby. Furthermore, the zeal of a specialist designer using only specific material products may result in innovations that would otherwise never have been conceived. There are many examples of this, and many famous engineers have become associated with particular materials of construction.

Consequently, there are today these separate engineering disciplines in modern bridge design and construction, often with very little cross-communication. Although historically structural steel had a big lead, the applications of concrete have steadily increased. In bridge design it can today provide better solutions in a majority of cases over a very wide range of circumstances, with the exception of very large spans exceeding at present approximately 400 m. In the general interest, however, it remains necessary that the solution of the problems posed by any project be given the widest possible consideration and that alternatives be considered, as discussed in Chapter 5, Section 5.9.

1.3 Modern Concrete Bridges: a Summary of the Different Configurations and Structural Systems

It is traditional to subdivide a bridge into (a) the superstructure, (b) the substructure, and (c) the foundations, as there is usually a clear division between these parts in slab, beam, girder, frame and truss bridges. This does not, however, always apply to arch-type and suspension or cable-stayed girder bridges.

For the purpose of classifying bridges, it is therefore more convenient to describe the configurations of the *primary structural systems* (Fig. 1.13) which are constituted of the main load-carrying members that support the *deck structural system* by spanning between the substructures or foundations. For the purposes of this chapter, the following definitions will apply:

(a) The *bridge deck structural system* is that part of the superstructure that directly supports and may be integral with the members that form the deck surface. It may be furnished with some of the following:

1. balustrades (parapets) and crash barriers;
2. highway surfacing, pedestrianways (footways) or raised sidewalks, traffic islands or railway tracks on ties and ballast;
3. expansion joints;
4. drainage systems;
5. service ducts or brackets carrying cables or pipes;
6. fixtures, brackets or recesses for supporting lighting poles or signboards.

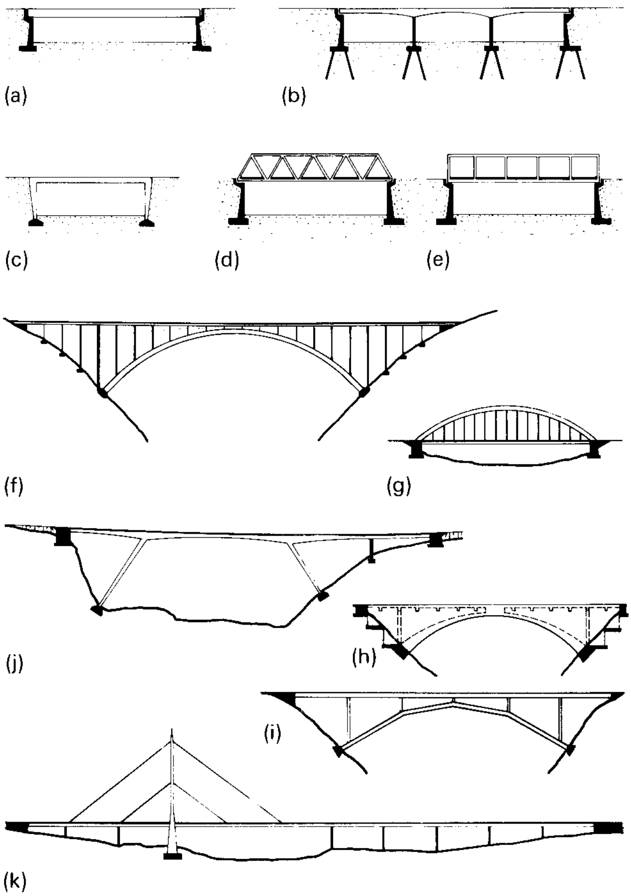


Figure 1.13 Some examples of primary structural systems. Group 1 (a, b) slabs, grids, beams and girders, and (c) portal frame; Group 2 (d) trusses and (e) Vierendeel girder; Group 3 (f) open spandrel arch, (g) tied arch, (h) solid spandrel arch, (i) funicular arch, and (j) strut frame; Group 4 (k) cable-stayed girder bridge

Items (1) to (6) are described in Chapter 5, Section 5.8.

(b) The *substructure* comprises those portions of the piers, columns, or abutments and their capping beams and bearings that act as direct support to the deck structural system.
(c) The *foundations* support substructures or, in the case of structures such as arches and cable-stayed bridges, provide direct support to the primary structural systems and may consist of concrete footings, spread foundations or rafts bearing directly on soil or rock, or capping slabs supported on caissons or piles.

Foundations and substructures are described in greater detail in Chapter 5, Section 5.4.

1.3.1 Configurations of Primary Structural Systems

Although there are countless variations in bridge types, they can broadly be classified into four groups if defined in terms of the configurations of the primary structural systems that mainly resist gravitational loads:

1. slabs, coffered slabs, grids, beams, girders, cantilevers and frames in which loading induces mainly bending, torsional and shear forces (Figs 1.13a–c);
2. trusses, including cantilevered trusses* and frames in which linear elements are joined together to form systems of interacting members in which either compressive or tensile extensional forces predominate (Figs 1.13d, e);
3. arches and related types in which the configuration and shape of the main load-resisting members are such that compressive forces predominate (Figs 1.13f–j, 1.27);
4. cable-suspended structures, i.e. suspension bridges and cable-stayed bridges in which tensile cables, supported over towers, form the main supporting system acting in conjunction with the deck structural system (Fig. 1.13k), and related types such as prestressed concrete ribbon bridges (Fig. 1.23).

Combinations of the above systems have been tried, but usually with unsatisfactory aesthetic results (see Section 1.4). This statement only refers to the primary structural role of these systems resisting mainly gravitational loads, as in most structures these systems also have 'secondary' roles in resisting, for example, wind forces such as the bow-girder action of an arch acting in conjunction with bending in the horizontal plane of the deck structural system. There are, however, exceptions such as the Vierendeel girder which combines system (1) and an incomplete system (2) to produce what is visually a very satisfying simplification. A good example is the 200 m (656 ft) span Rio Colorado Bridge in Costa Rica,[1.27] completed in 1972 (Fig. 1.14) which is a precast, post-tensioned, cable-suspended concrete bridge which combines systems (4), (3) and (1). The two inclined piers function as the towers for the suspended cables and, because of their inclination, they also provide beneficial arch and cantilever effects. Unlike a conventional suspension bridge, the roadway deck is situated above rather than below the cables.

The factors affecting the selection of the most suitable configuration for the primary structural system, the proportions of the structural members, the deck system and the reinforcing and/or prestressing systems, are referred to in Chapter 2, Section 2.5 and Chapter 5.

* Cantilever truss bridges are often considered as a separate group because of the significant era of long-span cantilever truss bridges. Trusses are sometimes classified with beams.

Figure 1.14 Rio Colorado Bridge, Costa Rica (courtesy of Professor T Y Lin)

1.3.2 Bridge Deck Structural Systems

In small- and medium-span structures, the deck which carries the highway or railway is usually the primary structural system in the form of various types of solid, voided or coffered slab (Figs 1.15a–e), or monolithic slab and beam constructions (Fig. 1.15f). Decks may be continuous or simply supported over more than one span and may be cantilevered with drop-in sections in one or more alternate spans (Fig. 1.15g). The spans may have various degrees of skew depending on the crossing.

Simply supported (non-continuous) deck structures extending over several spans require movement joints which may provide a poor riding surface unless costly joint systems are provided. In the case of slab and beam construction, this may be overcome by reducing the number of joints by making the deck slabs of groups of spans continuous over the supports (Fig. 1.15h). Provision must then be made for longitudinal expansion and contraction by using either flexible columns or sliding bearings. With this technique, prefabricated beams can be used where centering is not feasible, e.g. over traffic, water, etc., and where achieving continuity of the beams is difficult.

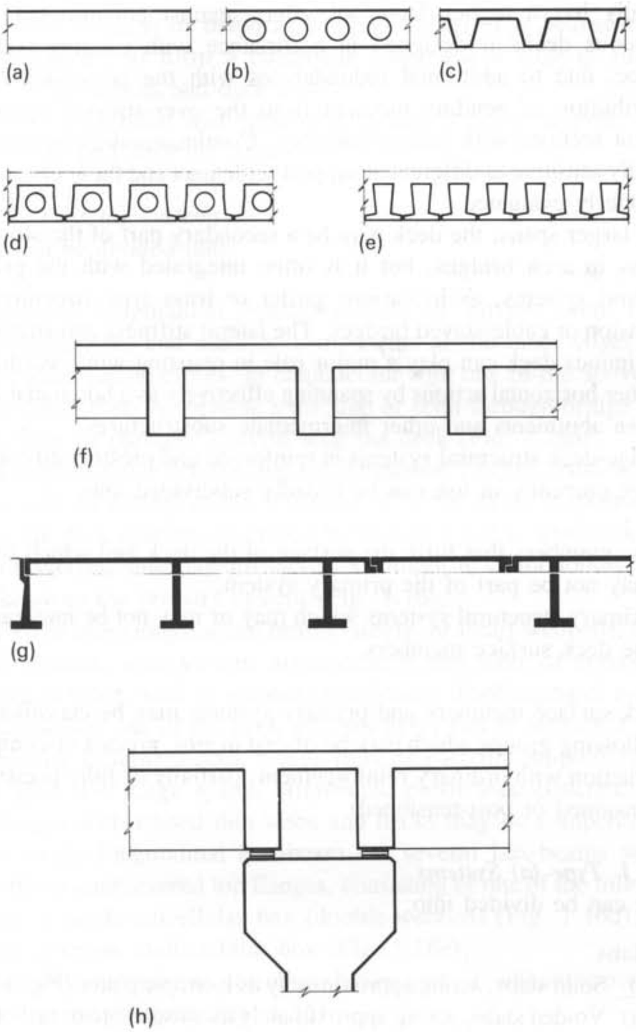


Figure 1.15 Bridge deck structural systems: (a) solid slab; (b) voided slab; (c) coffered slab; (d, e) contiguous voided or solid beams with reinforced concrete infilling; (f) monolithic slab and beam construction; (g) continuous deck with drop-in section; (h) detail of continuous slab with beam discontinuity

Drop-in spans provide a useful solution where expansion and contraction joints are required in a long continuous structure, especially if the movements cannot be accommodated at the piers or abutments. In continuous multispan bridges, longer spans are normally possible for the same construction depth as that for simply supported, non-continuous spans, due to the distribution of the bending moments. Haunched slabs or beams usually result in savings in materials, but may be undesirable for constructional, aesthetic or other reasons. It is usual to limit the end spans to approximately 85 per cent of the internal span length in order to equalize bending moments approximately. This type of structure normally has a higher factor of safety against collapse than non-continuous decks if designed in accordance with existing codes of practice, due to additional redundancies with the possibility of the redistribution of bending moment from the over-stressed section to adjacent sections with reserve capacity. Continuous-deck systems are normally sensitive to differential support settlement and these effects must be properly evaluated.

For larger spans, the deck may be a secondary part of the structure, such as in arch bridges, but it is often integrated with the primary structural systems, as in various girder or truss type structures and suspension or cable-stayed bridges. The lateral stiffness and strength of a continuous deck can play a major role in resisting wind, earthquake and other horizontal actions by spanning effectively as a horizontal girder between abutments and other intermediate substructures.

Bridge-deck structural systems in reinforced and prestressed concrete that are currently in use can be broadly subdivided into:

(a) the members that form the surface of the deck and which may or may not be part of the primary system;
(b) primary structural systems which may or may not be integral with the deck surface members.

Deck surface members and primary systems may be classified into the following groups which may be of cast *in situ*, precast or composite construction with ordinary reinforcement, partially or fully prestressed (pretensioned or post-tensioned).

1.3.2.1 Type (a) Systems

These can be divided into:

1. Slabs
   (i) Solid slabs, acting approximately as isotropic plates (Fig. 1.15a).
   (ii) Voided slabs, acting approximately as anisotropic or orthotropic plates (Fig. 1.15b).
   (iii) Coffered slabs, acting approximately as anisotropic or orthotropic plates (Fig. 1.15c).
   (iv) Precast beams of various cross-sectional shapes of constant or varying depth, contiguously joined together for the full depth by *in situ* reinforced concrete and transverse prestressing to effectively act as a voided or solid slab with the upper surface forming the deck (Figs 1.15d, e).

2. Beams
   (i) Longitudinal stringer beams with webs spaced apart integral with the deck slab which acts as an upper flange with or without

crossbeams or diaphragms (Fig. 1.15f). The beams may be precast to form a composite construction with *in situ* cast diaphragms and deck slab.

(ii) Longitudinal and transverse beams forming a grid system integral with the deck slab.

1.3.2.2 Type (b) Systems

These can be divided into:

1. Inverted longitudinal beams, trusses or girders with various arrangements and sizes of components at the outer edges of or between carriageways, in conjunction with one of the above deck systems, thereby forming a through or semi-through bridge which is fully or partially integral with the deck (Fig. 1.16a).
2. A single central longitudinal spine beam or T-beam, truss or girder of various arrangements and sizes of components supporting one of the deck systems described in Section 1.3.2.1. The deck spans transversely but may also act as a composite or monolithic upper flange to the primary system (Fig. 1.16b).
3. Two or more longitudinal beams (single- or multi-webbed), trusses or girders, with various arrangements and sizes of components spaced apart, with or without crossbeams, diaphragms or bracing, supporting one of the deck systems described in Section 1.3.2.1, thereby forming a deck bridge fully or partially monolithic (Fig. 1.16c). For large spans, orthotropic prestressed concrete girder bridges with ribbed thin webs and decks may be competitive.
4. A single longitudinal box-beam, or several box-beams with or without cantilevered top flanges, consisting of one of the following:
   (i) a single unicellular box (double-webbed) (Fig. 1.16d);
   (ii) a single multicellular box (Fig. 1.16e);
   (iii) twin or multiple unicelluar boxes with or without crossbeams or diaphragms (Fig. 1.16f);
   (iv) twin or multiple multicellular boxes with or without crossbeams or diaphragms (Fig. 1.16g).
5. Solid or voided slab deck supported directly on piers with or without haunches or drop-heads, e.g. Elztalbrücke built in 1965 (Fig. 1.16h).
6. Frames, which in this context will be defined as structures with more than one member, which members may be in one or more planes and which rely for stability on the flexural or torsional rigidity at all or some of the connections between members. Other connections may have various degrees of freedom. This definition does not include the case of strut-frames or funicular arches, which are classified under arch-type structures. There is a large variety of configurations of frames that are used either as secondary members

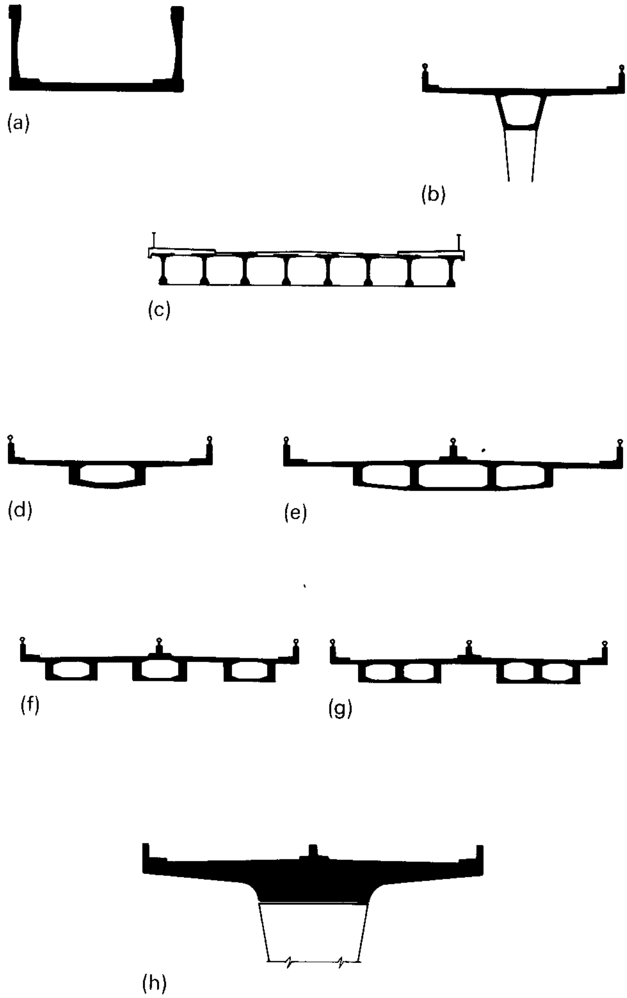


Figure 1.16 Primary structural deck systems: (a) through girder bridge; (b) spine beam; (c) longitudinal beams; (d) single unicellular box; (e) single triple-cellular box; (f) triple unicellular boxes; (g) twin double-cellular boxes; (h) solid deck

or as parts of the primary structural system, such as portal frames (single or multiple), Vierendeel girders, trestle piers, spill-through abutments and towers for suspension or cable-stayed girder bridges. Frames are generally suitable for short-span structures such as flyovers on freeways or bridges over small rivers, but some medium-span bridges such as the prestressed concrete portal frames designed by Freyssinet over the Marne have been competitive (see Section 1.2 and Fig. 1.5). Relatively flexible frames or articulated designs of statically determinate structures on bearings allowing rotation are suitable for sites where large differential settlements are anticipated.

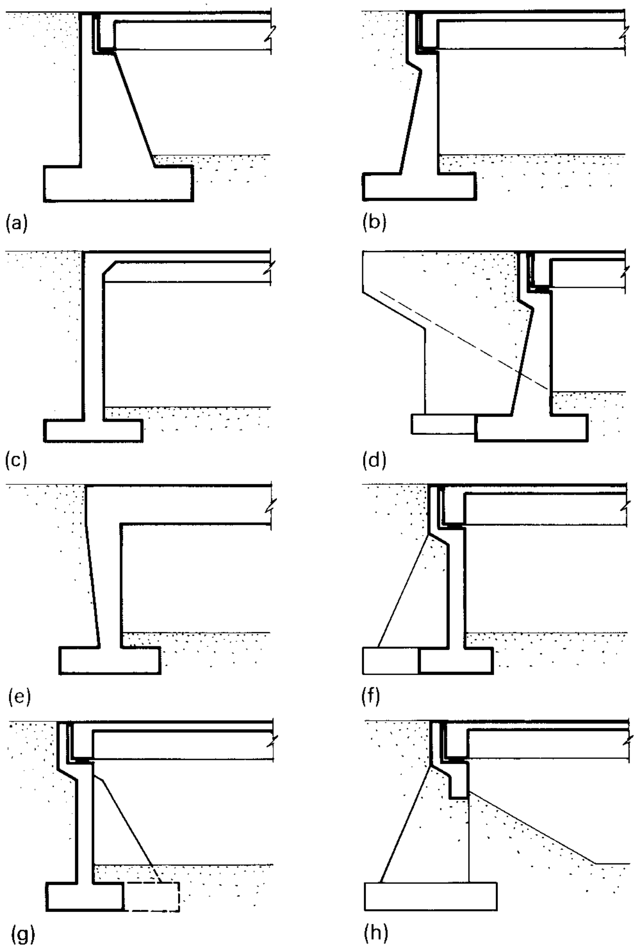


Figure 1.17
Abutment types:
(a) mass gravity;
(b) cantilever;
(c) diaphragm;
(d) wingwalls;
(e) portal frame;
(f) counterforted;
(g) buttressed;
(h) spill-through

1.3.3 Substructures Directly Supporting Deck Structures

At the ends of bridges, the deck structures are supported directly on abutments which can be of various types (Figs 1.17a–h) (see also Chapter 5, Sections 5.4.1 to 5.4.4):

(a) Mass concrete gravity abutment.
(b) Closed end abutments with:

1. solid or voided walls acting as cantilevers or struts or diaphragms or forming part of a cellular construction with wingwalls or side walls or part of a rigid portal frame;
2. counterforted walls;

3. buttressed walls;
4. combinations of (2) and (3).

(c) Open-end or spill-through abutments with breast (trestle) beams supported on columns.

Abutments may be of the seat type supporting the superstructure on bearings or they may be monolithic with the superstructure.

In multispan bridges, the intermediate piers (see Chapter 5, Section 5.4.5) can be of the following types:

(a) Solid or voided walls with or without capping beams.
(b) Single solid or voided columns with or without caps, or mushroom-type.
(c) Trestle piers or bents with or without capping beams supported on solid or hollow columns which can be of a great variety of shapes, depending on whether the bridge spans a river or gorge, or overpasses a roadway or railway, and on aesthetic considerations.
(d) Specially shaped columns of which there are a great variety, e.g. V-shaped, forked etc. (Fig. 1.18).

As with abutments, piers may be of the seat type with bearings, or monolithic with the superstructure. Substructures are supported on various types of foundation (depending on the nature of the subsoil), such as spread footings, rafts, caissons or piles, as referred to in Chapter 5, Section 5.4.

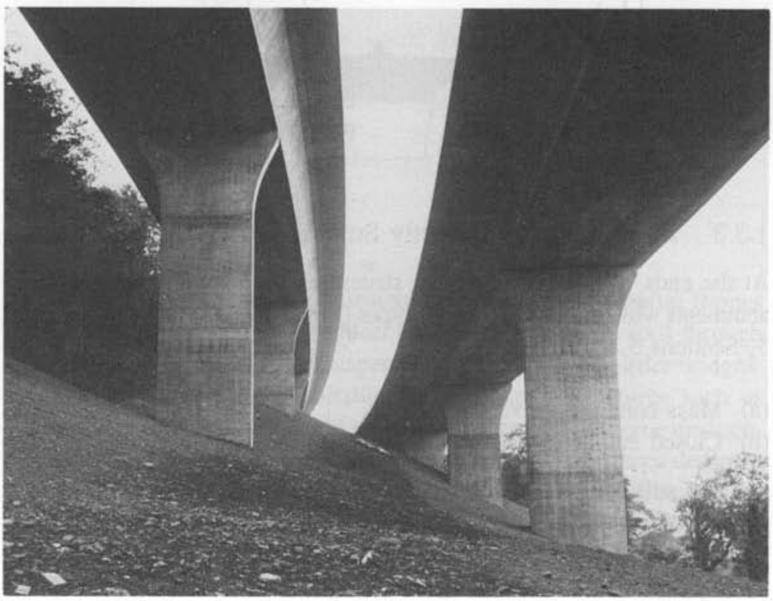


Figure 1.18 Wynhol Viaduct, Clevedon Hills, England (courtesy of Freeman, Fox and Partners)

1.3.4 Arches and Arch-Type Structures

With suitable foundation conditions, the arch has since early times (see Section 1.2) proved to be a competitive structural system in bridge engineering. With the correct profiling, symmetrically distributed loads such as dead loads induce mainly compressive forces, making concrete, with its high compressive strength/cost characteristics, an ideal construction material. Unsymmetrical loading due to traffic, as well as temperature effects and shrinkage and creep shortening effects (Chapter 5, Section 5.6), however, induce bending moments in the arch for which reinforcement must be provided. Due to the high compressive stress component, the amount of reinforcement required for the serviceability condition of the bridge is relatively low compared with other structural systems which rely mainly on flexure as a system of support.

In order to function as an arch, the foundations must be restrained from moving apart. For this reason, large arches require stable banks to provide this support. Smaller arch structures have, however, been constructed on relatively poor foundations using ties between the foundations to provide horizontal support. In arch bridges, the span-to-rise ratio is an important factor which controls the structural behaviour and economics of the structure and which must be carefully considered for each site, taking into account the cost and proportioning of other elements of the bridge.

There is a large variety of arch-type structures of which the following are representative:

1.3.4.1 True Arches

These may be classified as

(a) open spandrel arches (Fig. 1.13f);
(b) solid spandrel arches (Fig. 1.13h);
(c) tied arches (bow-string) which are usually through bridges (Fig. 1.13g);
(d) funicular arches (Fig. 1.13i).

Arch cross-sections may be of solid construction, but larger sections are invariably uni- or multi-cellular. Depending on the width-to-span ratio of the structure, arches may be single-, double- or multi-ribbed, with or without crossbeams and diaphragms.

Small arches have been built to circular, elliptical or parabolic profiles, but in modern times it is more usual to express the profile in a suitable mathematical form (usually a power series) that corresponds as closely as possible with the dead load thrust line. In the case of small arches, aesthetic requirements may dictate a deviation from the ideal profile. Arches may be encastred with no hinges, or may have two or three hinges, depending usually on the nature of the founding conditions.

Open spandrel arches are commonly used over rivers and deep gorges in the span range of the order of 100 m (328 ft) up to 300 m (984 ft). In the 1960s, many bridge designers favoured the newly developed cable-stayed structures in this span range because of the cost of temporary arch centering. The suspended cantilever method of construction developed in the late 1960s, which is referred to in Section 1.2 and described in Chapter 5, Section 5.6, has once more made arches competitive for specific sites such as deep gorges and sea straits. The new mainland to Isle-of-Krk Bridge near Zagreb in Yugoslavia has a span of 390 m (1280 ft). Solid spandrel arches are rarely built today, with the exception of small-span structures with a specific aesthetic requirement.

Tied arches are suitable as through bridges over rivers in the span range of the order of 100 to 200 m, where a maximum clearance for yachts or ships is required without raising the road or railway level unduly, or where relatively poor founding conditions do not provide adequate thrust resistance. Funicular arches may minimize the material costs, but are not necessarily the most economic and rarely provide a satisfactory aesthetic solution. Multiple arches have the advantage that dead-load thrusts are balanced, except at the end abutments.

1.3.4.2 Related Arch-Type Structures

A fifth type is the strut-frame (inclined leg frame) (Fig. 1.13). In the case of these structures, the thrust line deviates considerably from the profile of the linear structural members and bending forces are consequently a major consideration. Strut-frame bridges are often built as overpass structures over double carriageway highways where a central column is not desirable (Fig. 1.19) and have been constructed over deep gorges with spans approaching 200 m measured between the springing points of the struts. The prestressed concrete strut-frame bridge over the Gouritz River in South Africa has a clear span of 171 m (561 ft) between springing points (Fig. 1.20).

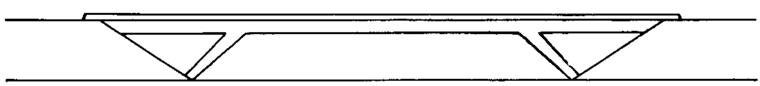


Figure 1.19 Strut-frame over freeway

1.3.5 Cable-Suspended Concrete Structures

1.3.5.1 Suspension Bridges

Modern suspension bridges with draped cables and vertical or triangulated suspender hangers are usually economical in the span range exceeding 300 m and are more competitive with the lighter steel deck constructions. The maximum span to date with a concrete deck is the Save River Bridge in Mozambique, with a span of 210 m (689 ft).[1.7] The towers,

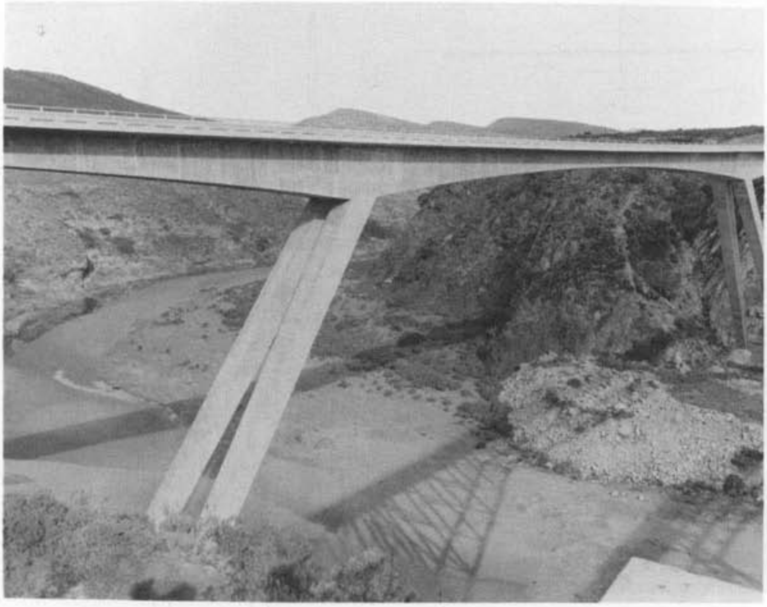


Figure 1.20 Strut-frame over Gouritz River, South Africa

substructures and minor approach spans are usually more economical in concrete. Concrete towers were used for the world's longest steel deck suspension bridge with a span of 1396 m over the Humber river in England.[1.23] Proposals for a 3000 m span bridge to cross the Messina Straits between Sicily and Italy have been considered.

1.3.5.2 Cable-Stayed Bridges

Modern cable-stayed bridges[1.24] are at present considered to be the most interesting development in bridge design. These structural systems are statically highly indeterminate and have become practically feasible by the development of sophisticated methods of analysis, coupled with the advent of improved electronic computers during the last three decades. The latest designs have extremely slender members and are competitive over the span range of the order of 100 to 700 m with concrete decks, and even greater spans in excess of 1000 m with steel decks. The towers, which are usually single or twin columns (pylons) or frames of various shapes including A-frames, are in most cases competitive in concrete. Examples are given in Section 1.2 and Chapter 5, Section 5.7. The cable arrangements are classified in terms of:

(a) the elevational arrangement, e.g. single, radiating, harp, fan and star, or combinations thereof, and may be asymmetrical about the pier depending on the relative span lengths (Figs 1.21a–f and 1.22);
(b) the transverse arrangement, e.g. single plane–vertical (central or

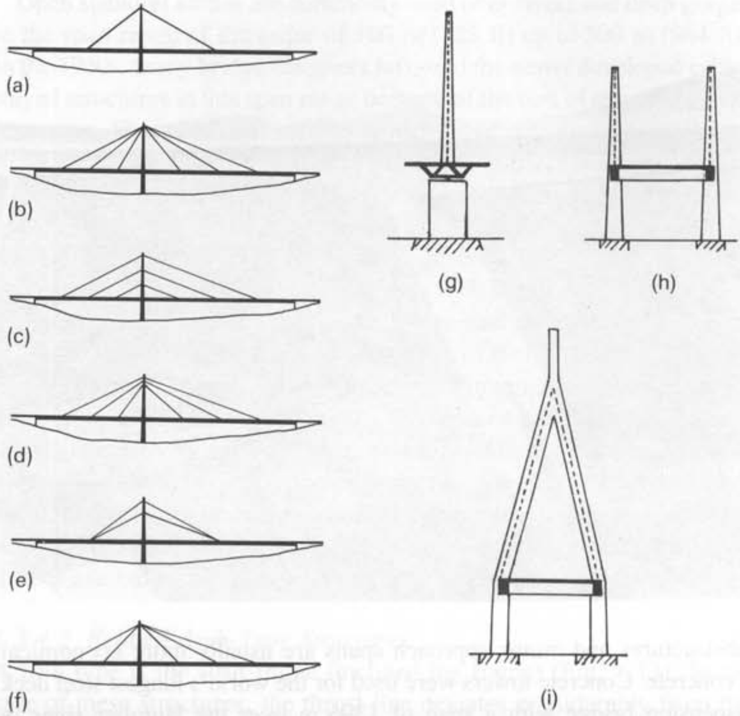


Figure 1.21
Cable-stay configurations: (a) single; (b) radiating; (c) harp; (d) fan; (e) star; (f) asymmetrical radiating (g–i) Typical cross-section arrangements

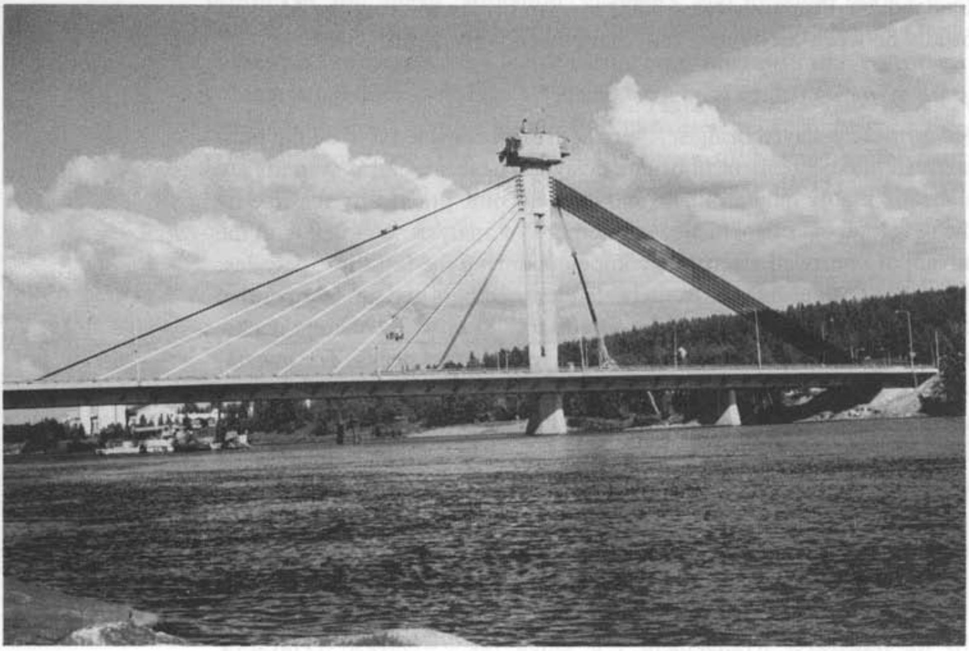


Figure 1.22
Kemijoki Bridge in Finland (courtesy of Dr P Marti, VSL International)

Figure 1.23 Maracaibo Bridge, Venezuela (courtesy of Professor Ing. Riccardo Morandi)

eccentric); double plane–vertical; or double plane–sloping (Figs 1.21g–i);

(c) the number of cables, e.g. single, double, triple, multiple or combined.

Although earlier designs were based on fewer support cables encased in concrete and spaced at greater distances apart (Fig. 1.23), modern trends are for more cables at closer spacing which allow a considerable saving in deck material. The cable configuration is dependent on the deck structural system (and vice versa) and the type of loading. Cables arranged in a single plane will generally require a torsionally stiff deck, whereas for cables in two parallel or sloping planes a solid slab may be adequate, depending on the width of deck (see Chapter 5, Section 5.7).

1.3.5.3 Combinations of Suspension and Cable-Stayed Bridges

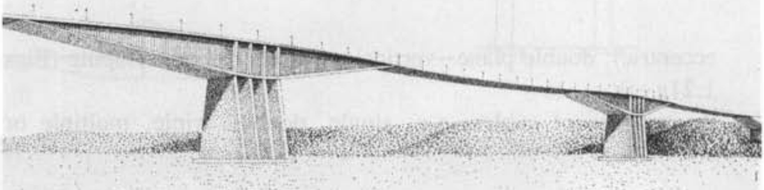
Examples of such combinations were discussed in Section 1.2 and have in more recent times been suggested by Dischinger[1.24] and also by the designers of the proposed Messina Straits Bridge.

1.3.5.4 Stressed Ribbon Bridges

These consist of prestressed concrete slabs or ribs with a sagging profile anchored into the abutments and subsoil or rock. The Genf–Lignon Bridge[1.7] has been successfully used as a pedestrian and pipeline bridge with a span of 136 m (446 ft) over the Rhône River in France. The longest span, however, is that of the bridge carrying a belt conveyor at the cement works of Holderbank–Wildegg in Switzerland, with a span of 216.4 m (710 ft) and a 14.75 m (48.4 ft) sag. It consists of steel cables with precast concrete deck elements clipped to them.[1.7]

A proposal for a 2 km (1.25 mile) long stressed ribbon bridge has been made by Dr Finsterwalder, to cross Lake Geneva. This bridge is a combination of suspended concave spans with a radius of about 2500 m (8200 ft) and supported convex parts near the piers, which have varying radii of 3000 m (9800 ft) or more. A limited prestressing of the stressed ribbon is accomplished so that the dead load does not produce tensile stresses.[1.28] The central and end spans are almost 460 m (1500 ft) long and alternate with 200 m (650 ft) spans. The stressed ribbon is a reinforced concrete slab about 254 mm (10 in) thick, with about 25 per cent of reinforcing steel. He also proposed a stressed ribbon design for the Bosporus Bridge (Fig. 1.24).

Figure 1.24 Stressed-ribbon bridge. Proposal for the Bosporus crossing by Dr Ing U. Finsterwalder



1.4 The Aesthetic Evaluation of Bridges

During the earlier periods in the development of bridge structures, especially masonry arches, the methods of construction were closely related to those for buildings. This probably accounts for the high standard in aesthetic design that was generally achieved. Michelangelo is recorded to have said: ‘A bridge ought to be built as though it were intended to be a cathedral, with the same care and the same materials.’ The significance of this statement is apparent if one bears in mind that bridge structures are visually prominent and often dominate the surrounding landscape.

It is quite apparent that the divorce of bridge engineering from architecture during the Industrial Revolution had a negative influence on bridge design. Some of the iron and steel bridges designed during the latter part of the 19th century indicated an almost total lack of sensibility to aesthetic considerations. There have, however, been many outstanding designs by engineers such as Telford, the Stephensons, Brunel, Roebling and Eiffel. Others after them, who were pioneers in the design of reinforced and prestressed concrete, such as Maillart, Freyssinet and several amongst the more recent designers, have not only set a very high standard of aesthetic design but have also had a significant influence on architecture. Although few engineers today would dispute the importance of aesthetics, they still generally have a singular lack of understanding of the subject and tend to see aesthetic design as a simple extension of engineering design. This fact can largely be attributed to the almost total separation of disciplines in most tertiary educational systems, which results in engineers very rarely receiving formal education or training in art or the appreciation of aesthetic values. This deficiency is compounded by the need for specialization and the consequential demands on the time of practising engineers in order to achieve excellence in the purely technical aspects of engineering.

Aesthetics, being a subject belonging to philosophy and the arts, differs essentially from the disciplines that constitute modern engineering. It follows that an understanding of aesthetics does not come naturally to most engineers. Fortunately, there are exceptions who, because of their domestic and scholastic backgrounds, through the influence of parents, friends or teachers, acquire an appreciation of art at an early age. Generally, however, engineers develop a predominantly logical approach to design without the intuitive sensibility and judgement that is essential for the appreciation and meaningful evaluation of the aesthetic aspects of their work. They have consequently over the years applied whatever innate abilities they may have had with greatly varying degrees of success.

The subject of aesthetics has, since the time of the earliest philosophers such as Plato, Aristotle and Plotinus, been the cause of much controversy and even today there is no universally accepted theory of aesthetics. The development of art and architecture has gone through many phases which also included various attempts at the formulation of principles and rules. These included the well-known rule of the golden ratio during the Grecian period, which was an age of formalism, and various subsequent attempts to produce geometrical formulations for beautiful forms or shapes. The 18th-century philosopher David Hume related beauty to utility. However, Emmanuel Kant was probably the first to assess aesthetics to be equal to and independent of reason and ethics. Most artists and architects today appear to agree that there are no rules by which one can create or measure the quality of art or architecture. Even if the relevant values were absolute in the Neo-Platonic sense, they would remain elusive to analysis in terms of our highly complex processes of perception. According to Herbert Read[1.37] in discussing the meaning of art:

> Many theories have been invented to explain the workings of the mind

> in such a situation, but most of them err, in my opinion, by overlooking the instantaneity of the event. I do not believe that a person of real sensibility ever stands before a picture and, after a long process of analysis, pronounces himself pleased.

Also, as Richard Padavan[1.38] has expressed it:

> We appreciate the proportions of a building instantly and intuitively, and not by first throwing an imaginary network of lines over it and then painfully calculating angles and ratios one by one.

Yet Roger Scruton[1.39] concludes:

> Like all decorative arts, architecture derives its nature not from some activity of representation or dramatic gesture, but from an everyday preoccupation with getting things right, a preoccupation that has little to do with the artistic intensions of romantic theory. My thesis has been that the aesthetic sense is an indispensable part of this preoccupation, and that the resulting 'aesthetics of everyday life' is as susceptible of objective employment as any other branch of practical reason.

Perhaps the most realistic 'working' definition for acceptable norms of aesthetic design is that which expresses them in terms of the 'average opinion' of a section of society that is constituted of leading professionals, practitioners, critics and other interested persons. Such 'common wisdom' which may be considered to represent 'good taste' is, however, not easily expressed in precise terms and will tend to vary in time and place.

Individual designers however, have, throughout history played significant roles in establishing styles that have become associated with good architecture of specific cultural periods.

Bridges in this context can be classified under architecture and any discourse on the aesthetic aspects of the latter would to a great extent apply to bridges which form a sub-class that are, however, less complex than most architectural creations. Bridge designers should therefore, to the extent that they are able, acquaint themselves with current trends in architectural thought relating to bridges. The most important attribute required of aesthetic appreciation and design is that of imagination of a rather special kind, which is probably inherited, but which can be substantially developed if nurtured by study, observation and much practice.

The type of mental activity that constitutes aesthetic judgement is probably not entirely unlike that which is necessary for innovative (creative) engineering design (see Section 1.6), which also requires an imaginative ability and is achieved largely by 'lateral' (non-logical) thinking.[1.40] 'Vertical' logic, which is suitable for applying or extending rules, is, however, essential for testing the validity of creative ideas. Innovative engineering design and aesthetic judgement are nevertheless different in essence as the latter is largely based on ideas selected from a wide range of impressions which cannot readily be quantified. An intuitive ability in the one field therefore does not necessarily imply an equivalent competence in the other.

However, according to Scruton[1.39] (*The Aesthetics of Architecture* © 1979 by Princeton University Press):

> Architectural tastes, in so far as they are tastes in the aesthetic sense, inevitably make way for deliberation and comparison. And here deliberation does not mean the cultivation of a vast and varied experience, like that of the over-travelled connoisseur. It denotes not the fevered acquisition of experience, but rather the reflective attention to what one has. A man might know only a few significant buildings — as did the builders of many of our great cathedrals — and yet be possessed of everything necessary to the development of taste. It suffices only that he should reflect on the nature of those choices that are available to him and on the experience that he might obtain.

Reasoning along similar lines one can surely presume that gifted bridge engineers can by reflective attention to their works, as did the above-mentioned builders of cathedrals, be possessed of everything necessary to the development of aesthetic taste.

There have always been gifted engineers with a good understanding of the subject, as was illustrated by a series of lectures on *The Aesthetic Aspect of Civil Engineering Design*, published by the Institution of Civil Engineers in 1944. In one of these. Sir Charles Inglis gave a very enlightening analysis[1.41] of the aesthetic qualities of bridges spanning a period of 2000 years, from the Pont du Gard in France (Fig. 1.1) to the Golden Gate Bridge in San Francisco. In his lecture, Oscar Faber[1.42] explained that beauty could not be derived from some simple short-cuts, but depended upon a large number of qualities of which the following are a few: harmony; composition; character; interest; the expression of function; the expression of construction; rhythm; colour; and texture of materials.

The famous Maillart (Section 1.2 and Figs 1.3 and 1.4) consulted no other designer, but practised three principles: to work within the constraints of the relatively new materials of reinforced concrete, to apply his original insight into deck-stiffened behaviour and to achieve minimum cost. There was no imposition of aesthetic rules in his designs, but there was a strong desire for aesthetic results.[1.43]

During the last decade or two, engineers have shown a renewed interest in aesthetics, coupled with an awareness of the need to relate their structures to the environment. This is reflected in numerous recent lectures and papers on the subject.[1.43–1.48] The general tendency is to formulate rules for aesthetic design. It is not the intention to decry the work of those that have in the past and recently produced such design rules, as there is no doubt that they serve a useful purpose as principles of selection and order. Even in architecture there are individuals and groups who are propagating theories bordering on rules, such as the writings of the architect-priest Hans van der Laan[1.49] with his concept of the Plastic Number, a proportionate theory which is founded on the 'way our intellect deals with the unbroken continuum of measures it encounters in nature'. He compares his concept of a ground-ratio expressed as a cubic equation for the three-dimensional world with that of the golden section, which can be expressed as a quadratic equation.

Previously le Corbusier (1948) had together with two young architects evolved the Modulor[1.50] as a tool which incorporates the golden section to determine proportion in architecture, sculpture and art. The basic dimension of the Modulor is that of a man 6 ft (1.83 m) tall.

It must be remembered, however, that most of these rules or laws, even those that are mathematically of considerable interest, have been deduced from empirical results and do not necessarily have a fundamental basis. They only work in an approximate manner in that they define some visual properties of objects that are aesthetically satisfactory and have withstood the tests of time. Generalizations should consequently be treated with great care. Every design can best be considered to be unique and, even where such rules are applied, an imaginative adjustment will invariably result in some improvement.

Rudolf Arnheim[1.51] has put forward a hypothesis based on Gestalt psychology that he claims goes a long way to explaining the mysterious working of architectural proportion. He argues that our sense of visual proportion arises from an innate faculty that operates by taking in the spatial field as a whole rather than reconstructing it piece by piece. This theory explains how we can judge spatial relations without measuring the lines or planes involved. Intuitive judgement, based simply on the inspection of a pattern as a whole, is assumed to rely on the strength and directions of the tensions experienced in the perceived object. It follows that the mathematical exactness of individual relations is less significant than the total pattern. Moreover the experience may even be enhanced by small deviations from exactitude such as occur in our natural environment.

Although feasible configurations of the structure of bridges are largely dictated by gravity and other forces and by the properties of the materials of construction, there is usually an almost unlimited number of possible variations in proportion and form that provide the designer with considerable scope for aesthetic improvement.

Most of the authors[1.44,1.52–1.54] who have suggested guidelines or rules are generally in agreement with the most important aspects. These are discussed below. It is not always easy to differentiate between the more generally accepted principles of composition and what may be mere working rules.

1.4.1 Unity of Form and Harmony

This is a prerequisite of all artistic or architectural works and even more so in the case of bridge structures. It is best achieved in modern bridges by simplicity of design and pureness of structure with continuous straight or smoothly curved lines and consisting of few and simple elements that are in harmony amongst themselves and with the environment. It requires the organization or arrangement of the members of a bridge in such manner that it excites within the observer a sense of wholeness generated by some central or dominating idea in the composition. Unresolved duality, such as that created by the voidal spaces in a two-span bridge (Fig. 1.25a), has a disturbing effect on the observer in that he has difficulty in finding a central focal point. A powerful motif created by increasing the visual mass of the central pier does tend to reduce the effect of the duality (Fig. 1.25b).

By going to the other extreme of expressing the deck and abutments more prominently than the central pier, as shown in Fig. 1.25c, the voidal duality may, however, be partially broken. Three-span bridges can be improved by making the central span longer than the approach spans (Fig. 1.25d). Four spans need careful treatment, as a form of duality can in some cases become apparent. The relations of pier dimensions to deck spans and depths and the treatment of the abutments all become important, as shown in Figs 1.25(a–d) and Figs 1.26, 1.27 and 1.28. In multispan bridges equal spans usually present few problems as any span becomes a minor component of the whole and the rhythmic repetition can be very satisfying. There is, however, the danger of such bridges creating a sense of boredom and some feature providing relief in the form of contrast may improve the design (Fig. 1.23).

The treatment of the abutments is very important. If they are too large in proportion to the other elements, they may mar the blending of the bridge with the banks of a gorge or the embankments of a roadway (Fig. 1.26). Side spans with smaller spill-through abutments are usually an improvement in roadway underpass bridges (Figs 1.27, 1.29 and 1.30). The shaping of the superstructure, especially the soffit and sides, is important. Shadows cast by projecting deck slabs may have marked effect on the visual form of the deck structure (see Figs 1.5, 1.20 and 1.36).

In addition to the above, the general problem of relating a bridge structure in a harmonious way to the environment needs careful consideration of its location and siting as well as that of the treatment of the approaching roads and embankments so as to avoid any unnecessary cutting or spoiling (Figs 1.31 and 1.32).

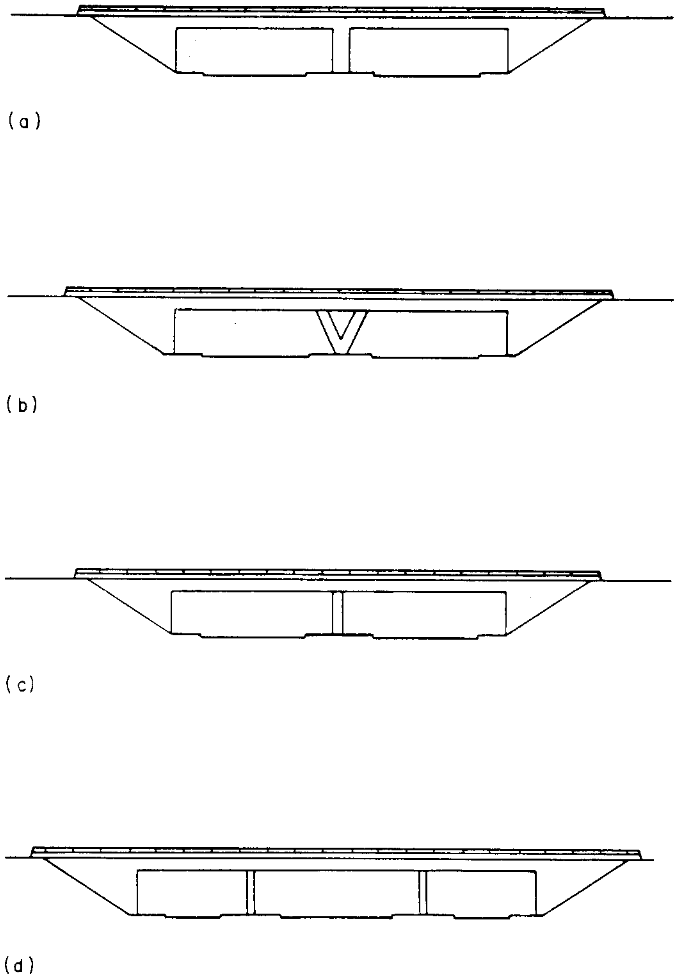


Figure 1.25
(a) Duality; (b) and (c) reduced duality; (d) effect of span ratios

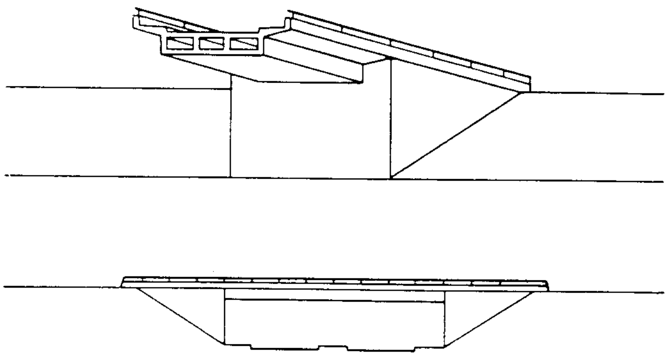


Figure 1.26 The effect of large abutments

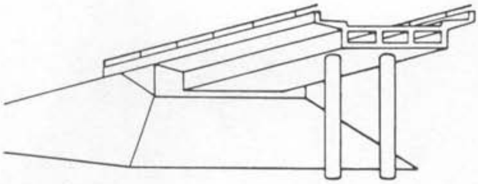
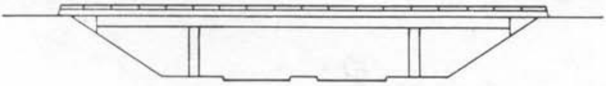


Figure 1.27 Spill-through type abutments



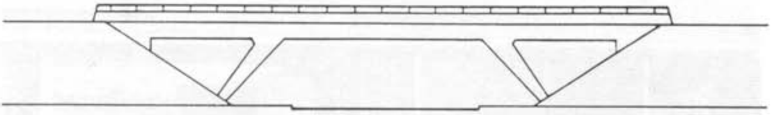


Figure 1.28 Negative spaces in strut-frame bridge

Figure 1.29 Abutment detail

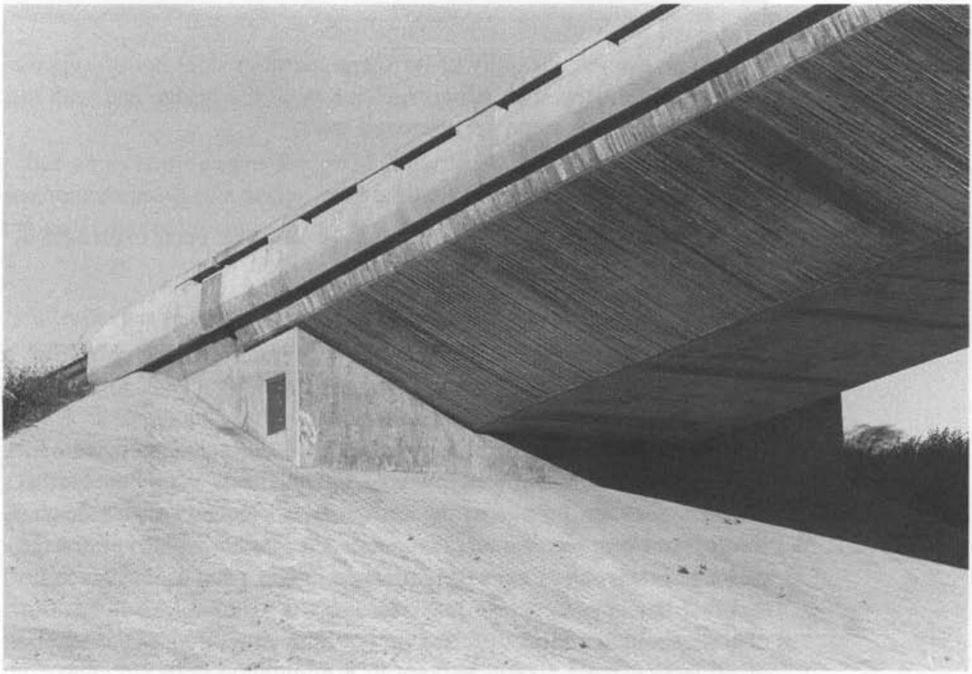


Figure 1.30
Motorway bridge in England (courtesy of Freeman, Fox and Partners)

The achievement of unity of form and harmony does not presuppose the exclusion of contrast between elements of the bridge and with the immediate environment, as discussed later.

1.4.2 Good Order

Good order is an aspect of unity of form and has been explained by Leonhardt[1.44] as

> . . . order of systems and order of direction of lines and edges. For bridges, order of systems means to choose a beam, an arch, a suspension bridge or a frame, but never to mix systems. Disorder cannot lead to beauty.

Whereas unity refers to wholeness, good order can be considered as one of the means towards attaining such wholeness. In practical terms, it means shaping, proportioning and aligning elements of the simplest forms possible in such manner that lines and edges are parallel or diverge, curve and intersect in a pleasing manner so that good order is manifest and that any appearance of haphazardness is avoided. A smooth flow of lines should connect different parts of the structure and the number of projecting parts should be kept to a minimum.

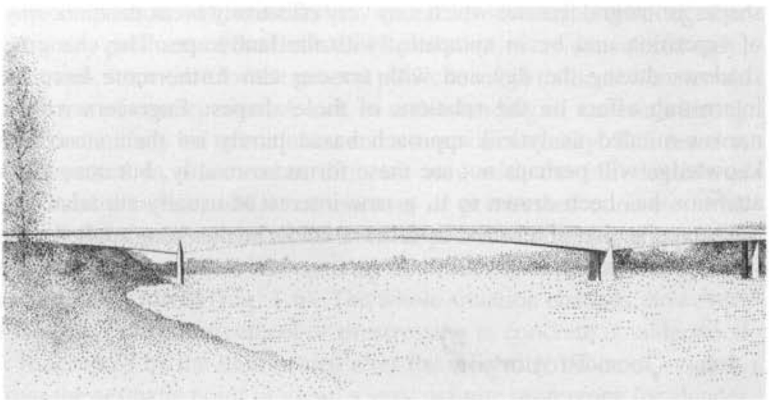


Figure 1.31
Bridge over the Rhine at Bendorf (sketch based on photograph in Ref 1.5)

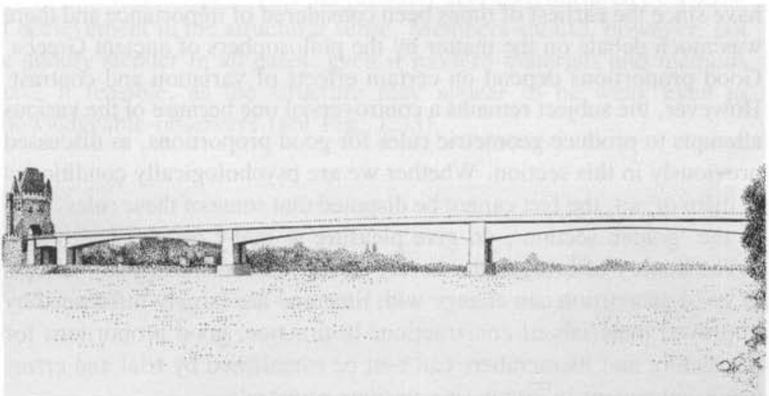


Figure 1.32
Bridge over the Rhine at Worms (sketch based on photograph by Dyckerhoff and Widmann)

But an excessive care for good order may result in dullness. The aesthetic meaning of a bridge can be enhanced by occasionally breaking the order to create contrast, as is discussed below.

1.4.3 Contrast in Form and Mass

Contrast is very important in expressing the content of an artistic work. In bridge design, it is equally so. A bridge consists of solid elements of various shapes and forms, contrasting also with open spaces or voids. The correct application of this concept of voidal or 'negative' spaces as a real part of the visual design, can add great interest and beauty and determine the essential character of a bridge (Fig. 1.4). A three-span strut-frame bridge over a gorge or freeway is a very good example of the central voidal space dominating the real masses, thereby enhancing the unity of design (Figs 1.20 and 1.28). The shaping of masses and the contrasting of negative spaces is thus very important because the discerning observer who looks at the bridge as a whole will read these shapes as integral features which may very effectively break the monotony of repetition and be in sympathy with the landscape. The changing shadows during the day and with seasons can furthermore have an interesting effect on the relations of these shapes. Engineers with a narrow-minded analytical approach based purely on their structural knowledge will perhaps not see these forms so readily, but once their attention has been drawn to it, a new interest is usually stimulated.

The application of textured surfaces or colouring has been applied with varying success.

1.4.4 Good Proportion

As described in Section 1.2, the proportions of the members of a structure have since the earliest of times been considered of importance and there was much debate on the matter by the philosophers of ancient Greece. Good proportions depend on certain effects of variation and contrast. However, the subject remains a controversial one because of the various attempts to produce geometric rules for good proportions, as discussed previously in this section. Whether we are psychologically conditioned to them or not, the fact cannot be disputed that some of these rules, such as the 'golden section', do give pleasure to many people.

The history of bridges, however, clearly demonstrates that concepts of good proportion can change with time and are largely influenced by improved materials of construction. In practice, good proportions for a structure and its members can best be established by trial and error, using judgement in comparing various proposals.

The proportioning of details requires great care, but it is the total effect achieved by combining members of various proportions that is most important. The ability to achieve a composition that is satisfactory comes only by practice and studying the works of others. A knowledge of art in general and its history of development is helpful to enable one to relate modern bridge design with older works that have been successful.

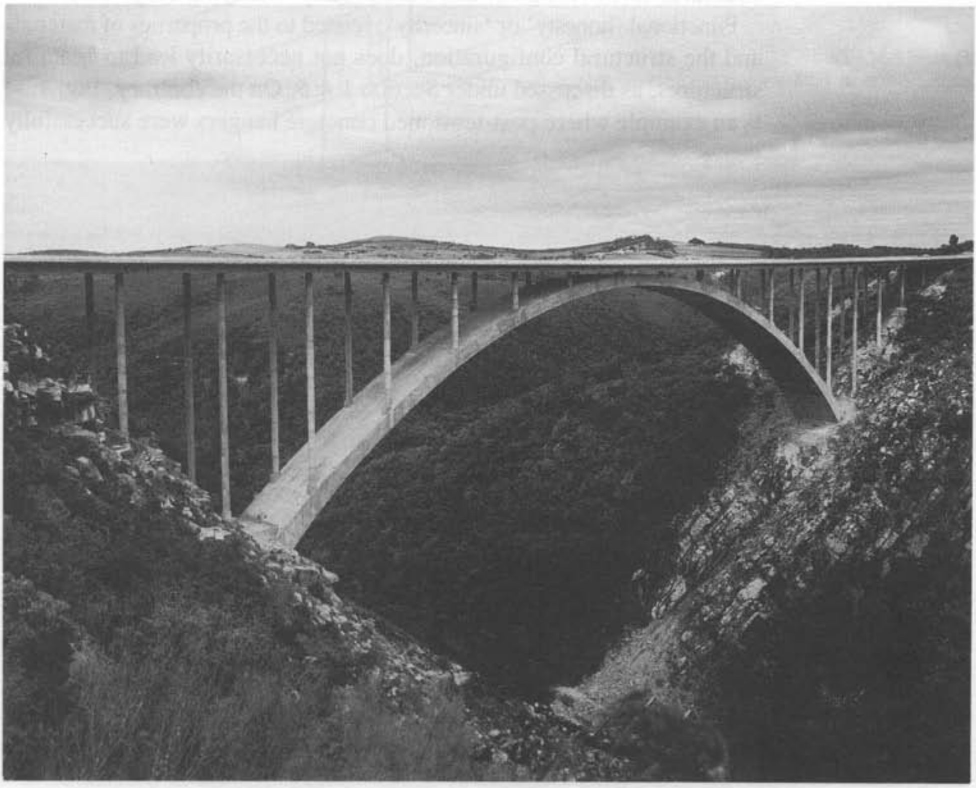
It must always be borne in mind that two-dimensional elevational drawings can be very misleading by comparison with our visual perception of the real bridge. There is always a degree of distortion of the perspective depending on the relative position of the observer.

1.4.5 Appearance of Strength and Stability

The users of bridges require to feel safe and consequently it must not only in fact be strong and stable, but it becomes an aesthetic requirement that its visual form must generate a sense of security by appearing to be so.

By comparison with stone arches, the earlier steel structures, together with many products of the Industrial Revolution, were considered ugly, and it took some time for the general public to adapt to the new materials and forms of construction. A lack of understanding of materials strong in tension initially led to confusion and a sense of insecurity. This reaction was repeated in a somewhat different way with the introduction of reinforced concrete, and even more so of prestressed concrete, with invisible reinforcement or tendons which provided no obvious evidence of how a brittle material like concrete of slender dimensions could resist large bending forces (Fig. 1.6). The whole situation changes, however, if the effect of reinforcement or prestressing in concrete is understood or is accepted by the uninformed after the reliability becomes evident. From the aesthetic point of view, a very definite preference for slender structures has developed. This has probably also been nurtured by a sense of achievement in the structural sense. Members should, however, not be unduly slender in all cases, even if modern materials and methods make it feasible, as the structure may appear to be weak even to knowledgeable observers (see Fig. 1.33).

Figure 1.33
Bridge over Van Staden's Gorge in South Africa



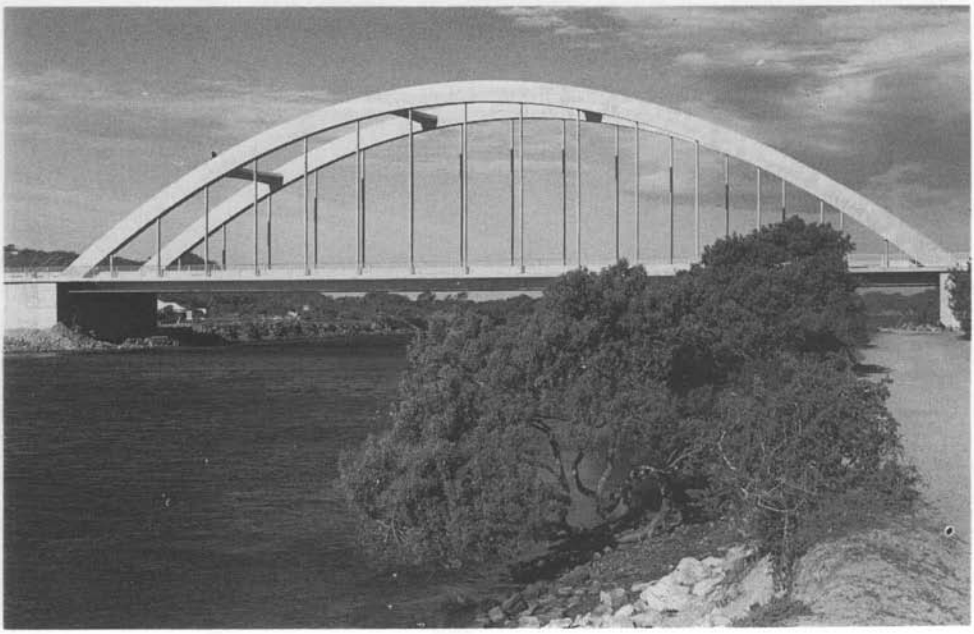
1.4.6 The Statical Form of the Structure Should Be Clearly Expressed

Provided care is taken to avoid the extremes sometimes understood under 'Functionalism', compliance with the requirements of Section 1.4.5 will normally dictate an articulated whole of interdependent structural parts. The structural requirements of strength and stiffness imply the equilibrium of forces and compatibility of deformations, thereby establishing a unifying law which contributes to the requirements of Section 1.4.1. The statical form can be emphasized to advantage by 'dissociating' certain parts that have different modes of structural function in resisting gravitational forces, even though they may interact effectively in resisting other actions such as, for example, wind and earthquake forces. Figures 1.11 and 1.33, which illustrate arches in which the decks have been clearly separated from the arches, are examples where this approach has been adopted with good results.

The Salgina Gorge arch designed by Maillart is a masterpiece in which an exactly opposite approach has been used to express the interaction between arch and deck in a smaller structure and in which the unified structural form is logical and aesthetically satisfying (Fig. 1.3).

Functional 'honesty' or 'sincerity', related to the properties of materials and the structural configuration, does not necessarily lead to beautiful structures, as discussed under Section 1.4.5. On the contrary, Fig. 1.34 is an example where post-tensioned concrete hangers were successfully

Figure 1.34 Tied arch bridge at Port Alfred, South Africa



used as tension members without belying their function. The bridge illustrated has a prestressed concrete deck and reinforced concrete arches.

1.4.7 Ornamentation

Ornamentation is seldom used in modern designs and is usually not required. It is, however, unwise to dogmatize that a bridge is only perfect if nothing can be omitted. Stark functionalism can be overdone. There are many good examples where some ornamentation has enhanced the beauty of a bridge and its sensible application must not be confused with misguided attempts to disguise the true function of structures. On the other hand, there are cases where it may be an improvement to disguise unsightly components, such as cable anchorages, bearings or ugly discontinuities in the structure (Fig. 1.35).

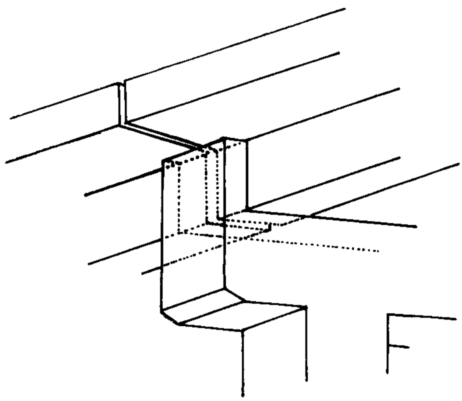


Figure 1.35 An example of an unsightly arrangement effectively disguised

1.4.8 The Need to Obtain Advice

The experience and insight that constitute a fully developed aesthetic sensibility is not readily attained. It is therefore imperative to consult an architect, artist or sculptor qualified and experienced in the design of solid forms, and preferably with some understanding of bridge structures and experience in the aesthetic design of bridges. As with most problems in engineering, the young enthusiast in bridge design may feel he also has all the necessary understanding to appreciate aesthetics. He would nevertheless be advised at least to work on the design with engineers more experienced in aesthetic design. In this connection, it must, however, be emphasized that the approach of designing a structure in accordance with engineering requirements and then attempting to improve the appearance by minor alterations or by ornamentation, is to be decried in the strongest of terms. A beautiful design can only be

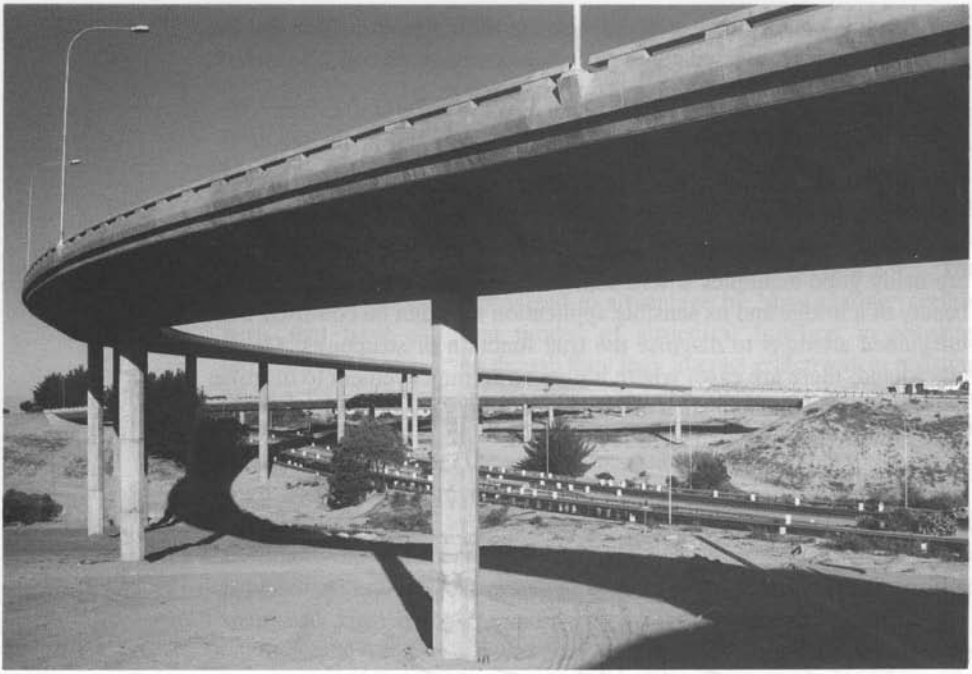


Figure 1.36
Motorway bridge near Cape Town, South Africa

achieved if the aesthetic design is developed as an essential part of the total concept (Fig. 1.36). There must therefore be a full involvement of the architect, or whoever is consulted, from the very beginning when the basic form is being conceived.

The above does not imply that individual engineers cannot develop a fully mature ability in aesthetic design. There are ample examples of engineering works which compare well with the best works that architects have produced.

1.5 Factors that Determine the Suitability and Competitiveness of Bridges

The suitability of a bridge is often evaluated in terms of the purely functional requirements or utility (see Chapter 3, Section 3.2), such as its reliability in the sense of performance, safety and durability, with total capitalized cost — including an estimate of probable maintenance costs for its serviceable life — being a prime consideration. As the functional requirements of a bridge can be fairly well defined in terms of specifications and drawings, the above-mentioned procedures are reasonably objective and have in the past been accepted as a satisfactory method.

Since the 1960s, people have become more aware of the need to preserve what is referred to as the 'quality of life', a term which is not easily defined, but amongst other things relates to the attainment of certain social and aesthetic standards and freedoms for mankind, while preserving as much of the beauty of the natural environment and its resources as is feasible and keeping it free of pollution. Likewise, engineers have come to recognize the importance and value of these considerations that extend beyond those directly related to engineering technology as practised in the past. Although there have always been exceptional designers who gave careful consideration to these matters, it must be admitted that this awareness has largely been generated by a strong reaction from various groups of nature-lovers, ecologists, architects and others who can in this regard collectively be classed as environmentalists, even although some of them have at times gone beyond reasonable bounds. It is nevertheless a fact that functional and economic considerations tend to predominate because of the prime needs of our society and it remains for us to do the best with whatever means are available to satisfy the broader requirements of aesthetics as discussed in Section 1.4, as well as complying with reasonable demands related to environmental preservation. Unfortunately, many of these considerations cannot be quantified accurately because of their subjective nature.

Various procedures have, however, been developed for doing so-called 'impact studies' to assess the effects of a project on the environment and the inhabitants of the affected area. Various authorities require Impact Statements which are usually considered by interdisciplinary committees prior to approval of the project. A discussion of the subject is beyond the scope of this chapter and reference should be made to published work.[1.55–1.62] It has in many cases been found that with proper preplanning and recognition of the above-mentioned factors, the overall design can be vastly improved with minimal extra expenditure.

It is consequently highly desirable that competitiveness should not be assessed entirely in terms of financial cost and that the concept of economy should be broadened to include these matters. The various procedures that are used for obtaining tenders or bids for proposed bridges, or for negotiating contracts, are given in Chapter 5, Section 5.10. These usually only make provision for obtaining a product that satisfies specified functional requirements, as defined above, which serve as the basis for comparative assessments of alternatives. In many cases, the other subjective factors discussed herein may tacitly play an important role.

There are further factors that may influence the choice of materials of construction that do not relate directly to the actual cost as contained in a tender, but that are considered beneficial to local industry by the responsible authorities. This is quite justifiable if economic considerations are seen in a broader context.

It is consequently necessary that all those factors that will be taken into consideration should be clearly defined and that methods and standards of assessment should be clarified. Where consultants are appointed, their brief should very clearly state the broader criteria of design. Consultants on their part should where necessary take the initiative in these matters.

1.6 The Conceptual Design Process

The conceptual design of engineering structures requires that the designer has a combination of mental attributes consisting at least of the ability to innovate by deductive and intuitive adaptation of existing concepts. In more imaginative cases, the conception of original ideas comes about by creative thinking. Although the nature of the mental processes of creative thinking or invention has largely been taken for granted and is even today not clearly understood, interest therein is not exactly a new development.[1.63] Initially it had mainly been the philosophers that had struggled with the problem. Some of the reasons for attention to the creative process were, however, practical, as insight into the nature thereof can increase the efficiency of almost any developed and active intelligence. Although logical thinking had since Aristotle been exalted as the one effective way in which to use the mind, this conclusion had been questioned for some time. Leibnitz (1646–1716) had expounded the concept of unconscious ideation. The notion of somewhat different mental processes that are not necessarily deductive or intutitive, and that involve an unconscious element in the inventive process, had already become well known in philosophical and literary circles in the early 19th century. However, it does seem that mathematicians have spoken of it in the clearest way, probably because in mathematics invention as a process is more easily recognizable.

When, at the beginning of the 20th century, Henri Poincaré gave his celebrated lecture[1.63] at the request of a number of Parisian psychologists to explain what in his personal experience invention was, he knew nothing of the findings of modern brain researchers. He said that the solution of a problem does not necessarily come about at the conclusion of a lucid and conscious effort but that, on the contrary — especially for the really difficult problems which led him to propose entirely new formulae, creative formulae one might say — the solution had surged forth when he least expected it, at times when he was doing something quite different. The role of what he then called the 'unconscious' is even more remarkable since, as he said, he was led to address himself without knowing why to a certain element of the problem, or to a difficulty which seemed to be without any relationship to the general problem with which he was struggling, as if for relaxation. Then, after days or weeks, he realized that what he had thought was a contingent phenomenon, was in fact precisely an element of the process of discovery which was to lead to the final solution.

The importance of the work of the unconscious in mathematical invention was thus clearly realized by Poincaré. On the topic of inspiration versus drudgery as the source of mathematical discovery, he concluded[1.64] that mathematical discoveries, small or great, are never born of spontaneous generation. They always presuppose a soil seeded with preliminary knowledge and well prepared by labour, both conscious and subconscious. A similar remark is attributed to Edison to the effect that genius is 99 per cent perspiration and only 1 per cent inspiration. However, 100 years before, Gauss had said:[1.65] 'I know that I discover things, but I don't know how I discover them, and when I reflect on it, I think that it can only be a gift from God, since things come to me all of a sudden without my having done anything, apparently, to merit them.' More recently, Professor Joseph Weizenbaum, discussing the work of psychologist Jerome Bruner, concludes[1.66] that we learn from the testimony of hundreds of creative people, as well as from our introspection, that the human creative act always involves the conscious interpretation of messages coming from the unconscious.

Henri Poincaré had also said about creative thinking[1.65] that: 'The important thing, if you want to find the correct idea, is to begin by thinking off-centre (*penser à côté*).' More recently Edward de Bono has developed the concept of lateral thinking[1.67] as an inductive method to develop new ideas and as a problem-solving technique that extends beyond logic. It employs a mix of random and logical procedures involving a certain amount of repetition, a certain amount of imprecision, all of which are inseparable from the process of bringing about a new idea. The complementary 'vertical' logic, which is suitable for deriving or extending rules or algorithms, is, however, essential for testing the validity of creative ideas in specific areas of engineering such as those related to the physical and functional aspects that influence structural reliability and effectiveness.

A comprehensive logical system in itself militates against innovation as rules negate the above-mentioned 'random freedom'. History is one long stream of examples that demonstrate this fact, as Paul Feyerabend has ably shown in his book titled *Against Method*.[1.68] He argues that the most successful scientific inquiries have never proceeded according to the rational method at all. He examines in detail the arguments which Galileo used to defend the Copernican revolution in physics, and shows that his success depended not on rational argument, but on a mixture of subterfuge, rhetoric and propaganda. Feyerabend argues that intellectual progress can only be achieved by stressing the creativity and wishes of the scientist rather than the method and authority of science.

Earlier, other philosophers like Popper[1.69] and Thomas Kuhn[1.70] had produced different arguments in which they demonstrated the limitations of the scientific method. Major advances in science, e.g. Newton's laws and theory of gravity, denied the logic within the accepted paradigm of that time and required *ad hoc* concepts, such as force acting at a distance, which defied all explanation. Modern science is no different and Max Jammer[1.71] gives an enlightening account of the conceptual development of quantum mechanics which reminds one in many ways of the discovery of the double helical structure of DNA by James D. Watson and Francis Crick, so humorously described by the former in his delightful book *The Double Helix*.[1.72]

In all these scientific works the importance of lateral thinking is predominant. Innovation in technology is a similar process. Established scientists were still proclaiming the impossibility of sustained flight by heavier-than-air craft when the Wright Brothers made their epoch-making flight at Kitty Hawk in 1903. Goddard experienced a similar resistance to his pre-war research in rocket flight and Whittle to his efforts to develop a jet fighter plane.

The underlying mental process in the innovative design of engineering structures is not unlike that in the other fields of creative effort mentioned above. It presupposes certain basic levels of knowledge and experience which are essential for the ability to apply the conscious and intuitive procedures, and a will to solve the problem, for the subconscious mental processes to culminate in ideas. P.R. Whitfield[1.73] has stated that, as a mental activity, the moment of creation appears to be largely outside our conscious control, although it is more likely to be stimulated when we have become immersed in a subject. A burning desire to find a solution, concentration, gathering and marshalling of facts and striving for completion by reaching out for still-vague ideas, are all activities we can feel and largely control at a conscious level. They mobilize and direct energy to finding a solution, but they are really only precursors to the act of creation, which seems to have a quality of spontaneity making it difficult to track and explain. Harding (1967) suggests[1.73] that the flash of inspiration often associated with scientific and engineering problems comes when the scientist tries to rest by turning away from his problem. When thinking or doing something else, the solution suddenly comes to him. Whitfield refers to the mysterious incubation phenomenon, which acts at a time of deliberate withdrawal.

In engineering, the expression of creativity is in part internal and personal and in part dependent on the external opportunities and pressures in an individual's environment. Creative, innovative and entrepreneurial aptitudes seem to need many strengths in addition to special talents in a particular field. Joint efforts by several individuals in the form of 'brainstorming' sessions have produced very fruitful results.

The adaptation of existing design concepts, configurations and details in design, to achieve the objectives and requirements of specific structural projects, constitutes a very large percentage of the work executed in practice and does not necessarily involve substantive innovation. However, much as it may conflict with the aspirations of the individual designer for a unique and novel solution, the mere reorganization of a design along the lines of existing works does not necessarily detract from the merits thereof. It may be preferable in economic terms to imitate or repeat successful designs than to invent purely for the sake of diversity.

The history of the design of engineering structures does, however, indicate that real progress is very largely dependent on innovative designs. Yet there are many aspects of the modern design process as practised that inhibit innovation. The underlying logic which forms the very basis of the design process is inherently restrictive on innovation. So also is an obligatory code of practice. The codification of precedures has become essential for good order and the standardization of methods is an objective that can be rationally justified in terms of sound economics, provided alternative procedures based on proven research are allowed.

Koestler (1964) observed that the act of discovery actually has a destructive and a constructive aspect; it must disrupt rigid patterns of mental organization to achieve the new synthesis. Only by escaping from the popular frames of reference and critically examining conventional methods and techniques can new ideas be developed and implemented. Disorder appears to be a necessary part of the creative sequence, and uncertainty goes with it.

Interesting as they are in suggesting how creative activity occurs, these observations offer little help in describing the actual process. We do not know what goes on at the neurone level, how nerve cells make their individual contribution or act together to form new patterns and insights. But there does seem to be a basic organizing and reorganizing activity going on all the time within the mind, which seems to select and arrange and correlate these ideas and images into a pattern. Innovation in engineering is therefore a complex problem-solving sequence which is not fully understood.

Judgement and approval of creative works by the general public is usually based on the 'common wisdom' of knowledgeable groups giving guidance. Engineering works are largely judged by their usefulness, but in structures aesthetics is important.

1.6.1 The Limits of Progress

Although there are apparently limitless possibilities of varying the detail of design conceptions by rearrangement of a particular structural configuration or fabric and changing the type and shape of its members, there do appear to be definite limits to significant progress in a more radical sense. It is almost impossible to give a clear definition of progress in general terms as it can mean many different things to different people depending on circumstances, but in structural engineering it can perhaps be most simply described in terms of the design criteria previously discussed, which are elaborated in the following sections. However, the measure of improvement, even for so practical a subject, cannot be absolutely quantified because of the inherent indeterminancy of some of those criteria.

Much has been written about the nature of progress and of future trends. The dynamics of progress, and their importance for the understanding of history, were set forth some 65 years ago by Henry Adams in his 'Law of Acceleration'. The acceleration can be explained in terms of reactions involving an element of positive feedback: the further the reaction has already progressed, the faster its further progress. But as Professor Gunther Stent[1.74] postulates:

> This very aspect of positive feedback of progress responsible for its continuous acceleration, embodies in it an element of temporal self-limitation. For since it seems *a priori* evident that there does exist some ultimate limit to progress, some bounds to the degree to which man can gain dominion over nature and be economically secure because of our boundaries of time, energy and intellect, it follows that this limit is being approached at an ever-faster rate.

There are many schools of thought on the general implications of this trend, varying from the pessimistic that believe that this limit will be reached soon, to others that optimistically consider such limits merely as thresholds to new developments generated by significant inventions.

In structural engineering there are obvious physical constraints that determine the bounds of the possible at any time. These bounds may be extended with the development of knowledge and new materials, but quite clearly they have limits which are related to the physical realities of the earth, such as the range of upper limits of the spans of various types of structures as determined by the weight and strength of materials of construction. For various forms and configurations of structure, these limits can be calculated using the materials or composites of materials that are available today. Galileo (in about AD 1600) came to the important conclusion that it was impossible to increase the size of structures to vast dimensions in such a way that their parts would hold together.[1.75] Super materials may extend these limits, but eventually upper limits will no doubt be reached.

Progress may also be approaching upper limits because of the apparent near exhaustion of ideas within the above-mentioned range of practical configurations. Some of these configurations were already foreshadowed in the earliest primitive constructions. The evolution of structures, as a process of sophistication of these configurations, has been largely related to the development and application of materials and methods of construction to meet specific needs.

The rates of progress in the various fields of application in structural engineering have in the past often been exponential, but usually reducing towards optimal ceilings or thresholds depending on whether or not pertinent ideas are expended, or whether subsequent innovations are of a sufficiently revolutionary nature to initiate new phases of development. New or improved materials and methods, often developing as a result of inventions in other fields, have generated innovation in structural engineering and created eras of rapid development. This happened during the Industrial Revolution and after the World Wars. Various benefits have been derived from by-products of space research programmes.

The state-of-the-art or philosophy of structural engineering has played a major role in determining the rate of progress. In the early days of the development of structures prior to AD 1800, design methods were largely intuitive, being based on experience (often catastrophic) and very elementary and rudimentary theory. In the early part of the 19th century, very significant advances were made in the theory of mechanics of materials by Navier (1785–1836), but it took several decades before engineers began to understand them satisfactorily and to use them in practical applications. This work heralded a new period in engineering and was probably the beginning of modern structural analysis. Navier was the first to evolve a general method of analysing statically indeterminate problems. His work was followed by major contributions of other famous mathematicians, scientists and engineers whose works have been well documented[1.6] and form the basis of modern structural engineering.

Today we are in possession of greatly enhanced empirical knowledge, coupled with the advanced methods of modelling and analysis provided by modern structural theory with powerful numerical methods used in conjunction with electronic computers, both for analytical work and computer-aided design. The modern design engineer is thus in a better position to evaluate alternatives and take decisions. His scope has widened considerably. Optimization and decision theories are paving the way to a better understanding of methods and procedures to realize objectives. However, there are limits to what computers can do,[1.66] and judgement will retain a most important role in structural design. This fact must be recognized as such in formal design procedures. Whereas the philosophy of cybernetics has had awe-inspiring success in its application to technological systems and in systems engineering, it is patent that the initial optimism with regard to automata with creative ability cannot be fully realized.[1.75]

1.6.2 Conception and Selection in Structural Engineering

1.6.2.1 General

A study of the historical development of bridges makes it very evident how various factors have influenced the selection of structural form in the past (see Sections 1.2 and 1.3). The fundamental basis has perhaps always been that of trial and error, from the primitive use of timber logs or boulders in crude masonry arches and ropes made of creepers or vines in small suspension bridges, to the sophisticated procedures used in the design of modern engineering structures.

It is clear that gravity and other forces due to loads and actions have played a major role in shaping bridge structures and determining the configurations. Experience gained in time, and lessons learnt from failures, have contributed to the knowledge that we have today. These, in conjunction with the theory of structures that has grown concomitantly with practical experience and experimentation, provide the basis for conception and selection in modern engineering practice. The process has become more sophisticated, but the role of intuition and unconscious ideation is as important as it was in the time of Leibnitz.

In form and configuration, the vast majority of innovative designs are rearrangements or adaptations of the fabric of proven designs. Such adaptations are often related to an improved understanding of loads and actions, usually based on theoretical analyses combined with experimentation, for example on wind forces and earthquakes and the response of structures thereto. Several notable innovations have been apparent, such as the improvement of the profiles of bridge decks of suspension bridges to reduce the wind effects, the elimination of gross movements due to earthquakes by special bearing arrangements, and increasing the ductility of piers and shear walls under extreme earthquakes.

The limiting trends referred to above imply neither that modern structural conceptions cannot be unique, nor that a major invention is not imminent. They only imply that the frequency of such events is reduced in well-established fields of structural engineering. I do not believe that bridge engineering has reached anywhere near the limits of excellence. In the application of materials and construction methods there has been a spate of inventions, although some of these were foreshadowed in other fields. There is also a definite trend towards improved methods of fabrication and control, resulting in better materials and improved structural performance and reliability whereby the designer's scope is increasingly widened. This process is bound to continue in the foreseeable future. The development of standardized design for economic reasons is not necessarily a limiting process.

1.6.2.2 Conception

Conceptual thinking is not necessarily confined to a single phase of the design process, but is essential to all the procedures for improvement. However, the initial ideas may be critical in setting objectives. Mentally, the designer should be attuned to a way of identifying the problems and seeking conceptual solutions that approximate roughly to the optimum. This comes from experience and a well-grounded understanding of how structures work; the ability to visualize the distribution of forces in structural members; to be able to assess the influence of the relative stiffnesses of members and the response to static and dynamic actions. The more refined that the designer's insight is, the sooner will the design process converge to effective and optimal solutions and the less likely will the occurrence of gross errors be.

A designer who has an understanding of the statistical properties of materials of construction, and of the indeterministic nature of the response of structures to random actions, will invariably be at an advantage to attain greater consistency in the reliability of the final product. This understanding should not only apply to the behaviour of individual structural elements or members, but to the assembly thereof, and the interaction amongst various components and the possible modes of failure. Risk is very much dependent on the combinatorial probabilities of failure of elements. Chain structures, with failure dependent on the weakest link, should if possible be avoided. This is mostly not possible in bridges, but then suitable adjustment should be made to safety factors where this is warranted. The converse applies where great redundancy is present. Similar arguments apply to single elements such as piers where the consequential damages of failure would be high. Such situations often occur during construction. First-level codes of practice do not allow for such discrepancies in risk, but a competent designer will take these effects into account (see Chapter 3).

Although the conception of new structural form is largely motivated by the need to solve engineering problems, the aesthetic aspirations of the designer are inseparably involved. The extent to which he succeeds in imparting visual quality to his works will depend on his sensibility to aesthetic values (see Section 1.4). The most successful designers of beautiful structural form clearly have a creative urge not unlike that of a sculptor. On structural projects such as bridges where visual form must come primarily from engineers, consultations with suitably experienced architects may nevertheless be beneficial. The modelling of form and configuration in this manner opens almost unlimited opportunities for aesthetic improvement by variation. This should not be confused with mere ornamentation. The various creations of Maillart and many others bear ample evidence of the ability of creative engineers to sculpt structural forms in a pleasing manner by going beyond pure functionalism in the process of solving specific engineering problems, but staying within acceptable economic bounds. Aesthetic design of a structure and its parts should therefore not be done as an afterthought, but should at all stages be part of an integrated process.

Designers tend to develop various optimizing techniques that either minimize internal energy, for example by using configurations or forms of structure that generate resistance by extensional forces in preference to bending, or by minimizing the response to actions, for example by designing shapes to reduce wind effects. Some would minimize materials or relate the design very closely to the construction methods. These objectives should not however be singled out.

The recent advances in methods of theoretical modelling and analysis and knowledge of structural mechanics, including the post-elastic and post-buckling phases, have opened new avenues of design and analysis which often extend beyond the reach of intuitive insight. Methods such as finite element analyses have become extremely powerful tools to achieve accurate simulation of complex structural behaviour. Conceptual design has thus gone a full circle and has reverted to a trial-and-error process of a nature which would have been impossible without the modern generation of computers at the levels of complexity to which we are referring. In design practice things generally happen more crudely, but the benefits of the results of the more sophisticated analyses are usually passed on to set new standards. There is a better perception of the statistical nature of actions, such as for example the structure of wind and the nature of earthquakes, and the response of structures thereto. However, problems in predicting certain trends, such as the modelling of traffic loading on highway bridges, which is not a purely random phenomenon but subject to human manipulations, have once again become evident. Authorities and experts in various countries still differ greatly on modelling of highway traffic (see Chapter 2, Section 2.4.3).

1.6.2.3 Selection

Selection is a very important part of structural design and consists of a search for optimal solutions by identification of possibilities, followed by evaluation and comparison, leading to the final choice. Whereas classical optimization procedures have limited application in structural design, numerical methods have opened new approaches. However, judgement still plays an important role in practice. Essentially, the decision-making process takes two forms. Firstly there are procedures for finding the best solutions for particular members or configurations of members, which usually consist of the step-wise or incremental adjustment of dimensions or forms in precalculated or random directions to obtain optimal solutions. Classical and numerical procedures can be applied in some of these cases. The other method distinguishes between alternatives that differ discretely or absolutely with respect to the parts or the whole, such as alternative designs with different configurations or of different materials. The basis of selection should be structural utility, as defined in Chapter 3, Section 3.2.1, even if in practice it can only be partially done by value analysis in terms of monetary costs with a qualitative assessment of other equally important but subjective criteria such as aesthetics and environmental impact (see Sections 1.4 and 1.5, Chapter 3, Section 3.2).

Although no part of the design process is unimportant, the choice of structural concepts is crucial. It challenges all those inherent and acquired abilities by which a designer takes decisions that determine the essential quality of an engineering structure. Although computerization is reducing the role of human designers in analysis and in the production of documentation, conceptual design will remain the domain of the engineer and well-designed engineering structures will therefore always bear the stamp of individual designers.

References

1.1 ISO 2394: *General Principles for the Verification of the Safety of Structures* International Organisation for Standardisation, Geneva, 1973, 5 pp

1.2 Comité Euro-International du Béton (CEB) 1978 *International System of Unified Standard Codes of Practice for Structures* vol I: *Common Unified Rules for Different Types of Construction and Material* 123 pp, and vol II: *CEB–FIP Model Code for Concrete Structures* Bulletin d'information, N.124/125-E, 348 pp

1.3 Hopkins H J 1970 *A Span of Bridges* David and Charles, Newton Abbot, 288 pp

1.4 Smith H Shirley 1964 *The World's Great Bridges* rev edn, Phoenix House, London, 250 pp

1.5 Wittfoht H 1972 *Triumph der Spannweiten* Beton Verlag, Düsseldorf, 314 pp

1.6 Timoshenko S P 1953 *History of Strength of Materials* McGraw-Hill, New York, 452 pp

1.7 Stephens J H 1981 *The Guinness Book of Structures* Guinness Superlatives, London, pp 11–60

1.8 Pevsner N 1968 *Pioneers of Modern Design* Penguin, London, pp 126, 183

1.9 Pugsley Sir Alfred (ed) 1980 *The Works of Isambard Kingdom Brunel* Cambridge University Press, Cambridge, pp 163–82

1.10 Jackson D C Oct 1977 Railroads, truss bridges, and the rise of the civil engineer *Civ. Eng. ASCE*, pp 97–101.

1.11 Williams J W Jr Oct 1977 James B Eads and his St Louis bridge *Civ. Eng, ASCE* pp 102–6

1.12 Kong F K, Evans R H, Cohen E, Roll F (eds) 1983 *Handbook of Structural Concrete* Pitman, London

1.13 Hamilton S B 1956 *A Note on the History of Reinforced Concrete in Buildings* National Building Studies special report no 24, DSIR, HMSO, London 30 pp
1.14 Robertson H 1952 *Modern Architectural Design* Architectural Press, London, pp 29, 38, 127
1.15 Siegel C 1962 *Structure and Form in Modern Architecture* Crosby Lockwood, London, pp 160–7
1.16 Richards J M 1965 *An Introduction to Modern Architecture* Pelican, London, pp 134–5
1.17 Menn C 1979 Fifty years of bridge building in reinforced concrete *Proc. Int. Ass. Bridge Struct. Eng. symp. on bridges* Zürich, 1979, pp 29–40
1.18 Freyssinet E Feb 1950 Prestressed concrete: principles and applications *J. Inst. Civ. Eng* **33**: 331–80
1.19 Freyssinet E 1954 *The Birth of Prestressing* Cement and Concrete Association, London, Library Translation no. 59
1.20 Virola J Apr 1969 The world's greatest cantilever bridges *Acier–Stahl–Steel* no. 4: 164–70
1.21 Virola J Nov 1971 The world's greatest steel arch bridges *Int. Civ. Eng* **II**(5): 209–28
1.22 Wex B P 1979 Case study — the Humber Bridge *Proc. Int. Ass. Bridge Struct. Eng. Symp. on Bridges* Zürich, 1979, pp 109–20
1.23 Wex B, Stephens J, Wood B, Glen H May 1981 Humber Bridge *New Civ. Eng. Supplement* 28 pp
1.24 Podolny W Jr, Scalzi J B 1976 *Construction and Design of Cable-Stayed Bridges* Wiley, New York, 506 pp
1.25 Podolny W Jr 1981 The evolution of concrete cable-stayed bridges *Concr. Int.* **3**(8): 35, 43–7
1.26 De Ing G A (Presenter) 1979 Il Ponte di Brotonne sulla Senna tra Rouen e Le Havre *L'Industria Italiana del Cemento* **9**: 499–518
1.27 Scordelis A C, Lin T Y 1977 Cable-suspended concrete structures *Proc. Conf. Int. Ass. Shell and Spatial Struct.* Alma Ata. 13–16 Sept 1977, 23 pp
1.28 Finsterwalder U 1965 Prestressed concrete bridge construction *J. Am. Concr. Inst. Proc.* **62**(9): 1037–46
1.29 Billington D P Sept–Oct 1976 Historical perspective on prestressed concrete *J. Prestressed Concr. Inst* 48–71
1.30 Candrlic V 1981 Cantilever construction of large concrete arch bridges in Yugoslavia *Concr. Int.* **3**(8): 43–7
1.31 Liebenberg A C 1980 Highway bridge over the Bloukrans Gorge — the longest arch bridge in Africa *Concrete/Beton CSSA* no. 18: 13–16
1.32 Report of the Royal Commission into the failure of West Gate Bridge, Government Printer, Melbourne, Australia, 1971, 143 pp
1.33 The modern design of wind-sensitive structures, *Proc. Seminar of Institution of Civil Engineers* London, 18 June 1970, Construction Industry Research and Information Association, London, 1971, 139 pp
1.34 Hay J S 1980 *An Introduction to Some Current Theories on the Aerodynamic Behaviour of Bridges* Transport and Road Research Laboratory, Dept of Environment, Dept of Transport, London, supplementary report 542, 50 pp

1.35 Simiu E, Scanlan R H 1978 *Wind Effects on Structures: an Introduction to Wind Engineering* Wiley, New York, chapter 6
1.36 Biétry J, Sacré C, Simui E 1978 Mean wind profiles and change of terrain roughness *J. Struct. Div. Am. Soc. Civ. Eng.* **104**(10): 1585–93
1.37 Read H 1949 *The Meaning of Art* Penguin/Faber & Faber, London, p 29
1.38 Padavan R Apr 1986 A necessary instrument? *The Architect*, London, p 29
1.39 Scruton R 1969 *The Aesthetics of Architecture* Methuen, London; Princeton University Press, Princeton, NJ 259 pp
1.40 De Bono E 1969 *The Mechanism of Mind* Penguin, Harmondsworth, pp 236–45
1.41 Inglis Sir Charles 1945 *The Aesthetic Aspect of Civil Engineering Design* (3rd lecture) Institution of Civil Engineers, London, pp 43–9
1.42 Faber O 1945 (1st lecture) Institution of Civil Engineers, London, pp 1–23
1.43 Billington D P 1980 Bridge aesthetics: 1925–1933 *Final Report 11th Congr. Int. Ass. Bridge Struct. Eng* Vienna, Aug–Sept 1980, pp 47–52
1.44 Leonhardt F Feb 1968 Aesthetics of bridge design *J. Prestressed Concr. Inst.* **13** (1) 19 pp (reprint)
1.45 Harley-Haddow T 1981 Structures in architecture *Struct. Eng* **59A**(6) 193–201
1.46 Slater R E 1980 Bridge aesthetics *Final Report 11th Congr. Int. Ass. Bridge Struct. Eng.* Vienna, Aug–Sept 1980, pp 115–20
1.47 Murray J Nov 1981 Visual aspects of motorway bridges *Proc. Inst. Civ. Eng.* **70**(1): 755–78
1.48 Elliott A L Jun 1968 Aesthetics of highway bridges *Civ. Eng. ASCE* 64–6
1.49 van der Laan H 1960 *The Plastic Number* Referred to by Padonvan R May 1968 Measuring and counting, *The Architect* London, p 25
1.50 le Corbusier (Jeanneret-Gris) 1955 (English edn 1958) *Modulor 2* Faber & Faber, London, p 335
1.51 Arnheim R 1955 *A Review of Proportion* Referred to by Padavon R in Ref. 2.35
1.52 Tarnos T P 1980 Relativity and optimization of aesthetic rules for structures *Final Report 11th Congr. Int. Ass. Bridge Struct. Eng.* Vienna, Aug–Sept 1980, p 59
1.53 Schmid W A 1980 Proportionen in de Natur und im Menschenwerk — wir messen, sehen und hören *Final Report 11th Congr. Int. Ass. Bridge Struct. Eng.* Vienna, Aug–Sept 1980, p 69
1.54 Yasuji T, Yoshio N On the manual for aesthetic design of bridges *Final Report 11th Congr. Int. Ass. Bridge Struct. Eng.* Vienna, Aug–Sept 1980, pp 101–8
1.55 Pearson J R Oct 1973 Impact statements — present and potential *Engineering Issues* pp 449–55
1.56 Bella D A, Williamson K J Jul 1978 Interdisciplinary research and environmental management *Engineering Issues* pp 193–202
1.57 Committee on Impact Analysis of the Water Resources, Planning and Management Division, ASCE Nov 1978 The civil engineer's responsibility in impact analysis *J. Water Resour. Plann. Mngmnt Div. Am. Soc. Civ. Eng.* **104**(WR1): 253–63
1.58 Jones L R May 1975 Environmental impact reports, problems and

opportunities *Civ. Eng. ASCE* pp 71–3
1.59 Wanielista M P, McLellon W M, Smith L L Jul 1975 Environmental impact statements and civil engineering *Engineering Issues* pp 321–7
1.60 *Ad hoc* Committee on Community and Environmental Problems Associated with Urban Highway Proposals, *Report* National Association of Australian State Road Authorities, 1974, 49 pp
1.61 Urban Transportation Division, ASCE 1973 *Proc. Environ. Impact Spec. Conf.* Chicago, 21–23 May 1973, American Society of Civil Engineers, New York, 393 pp
1.62 Weck T 1977 Environmental impact of transportation projects *Environmental Impacts of International Civil Engineering Projects and Practices* American Society of Civil Engineers National Convention, San Francisco, 17–21 Oct 1977, pp 1–28
1.63 Ghiselin B (ed) 1952 The creative process, a symposium. In *Mentor* New American Library, New York, pp 11, 33–42
1.64 Bell E T 1953 *Men of Mathematics* vol 2, Pelican, Penguin Books, London, p 604
1.65 Moraze C 1977 Literary invention, the structuralist controversy. In Macksey R, Donato E (eds) *Proc. Symposium* Johns Hopkins University Press, London, 3rd printing, pp 23, 35
1.66 Weizenbaum J 1976 *Computer Power and Human Reason* W H Freeman, San Francisco, pp 218, 221
1.67 De Bono E 1977 *Lateral Thinking* Pelican (reprint 1980), London, p 260
1.68 Feyerabend P 1979 *Against Method* Verso, London, p 339
1.69 Popper K 1967 *Logic of Scientific Discovery* 2nd edn, Hutchinson, London
1.70 Kuhn, T 1962 *The Structure of Scientific Revolutions* University of Chicago, Chicago
1.71 Jammer M 1966 *The Conceptual Development of Quantum Mechanics* McGraw-Hill, New York, pp 399
1.72 Watson, J D 1970 *The Double Helix* Penguin, London p 175
1.73 Whitfield P R 1975 *Creativity in Industry* Penguin, London, p 217
1.74 Stent, G S 1978 *Paradoxes of Progress* W H Freeman, San Francisco, p 231
1.75 Rose J (ed) 1969 *Survey of Cybernetics* Illife, London, p 27

2 Design Criteria for Bridges

2.1 General

In order to carry out a brief for the design of a bridge, it is necessary that all design criteria be clearly defined or established. The extent of the designer's duties and responsibilities will depend on the information provided by the client body. This can vary from a bare minimum to elaborate codes of practice and design manuals as well as standard documents of general conditions of contract, general specification clauses, tender documents and procedures related to contract administration and supervision. As the standards applied by various authorities in different countries vary greatly, general principles only will be covered in this section, with reference to specific documents where it is necessary to illustrate certain points.

2.2 Class of Bridge

Bridges can conveniently be classified in terms of their functional purpose, the configuration and structural systems, the materials and methods of construction used and the nature of the site. Under the heading of 'functional purpose' would be indicated: the type of traffic, for example, class of highway, railway, airport runway, cycle or pedestrian traffic and the applicable geometric standards of the roadway; the type of pipeline or conveyor; the nature of the space that it must span, for example an underpass bridge spanning a number of railway lines and service roads, or a bridge spanning a deep gorge or a wide river in flat terrain. Under 'configuration', a description of the primary structural system that resists gravitational loads is usually adequate, but it may be necessary to elaborate the detail of secondary structural systems (see Chapter 1, Section 1.3). Under 'materials of construction', the main classifications for concrete bridges would be ordinary reinforced concrete, prestressed concrete or composite. The type of reinforcement, the type, method and degree of prestressing would be indicated as well as whether all the concrete is cast *in situ* or, if not, which elements are precast. If composite, the nature of the other material would be described.

The 'method of construction' (see Chapter 5, Section 5.3) is often important in bridge classification as well as the features of the site that would affect the design and construction method (see Section 2.3).

2.3 The Nature of the Site, Topographical, Environmental and Foundation Conditions

The broader implications of the impact of a bridge structure, and the road system of which it is part, on the environment have been referred to in Chapter 1, Section 1.5. From a purely engineering point of view, one cannot overstress the importance of a proper inspection, survey and investigation of the site topography and subsoil conditions and of the probable hydraulic behaviour of a canal, river or estuary (where applicable). The malfunction or failure of many bridges in the past has been attributed to inadequate site investigations.

The practical procedures that are usually adopted for the selection of the most suitable site are described in Chapter 5, Section 5.2. The actions that may be imposed on the projected structure will depend on the class of bridge and the site location, which will determine the probable nature and magnitude of actions due to natural causes. These are described in Section 2.4.

The soil or rock on which the structure is founded, or which is retained by parts of it, should be considered as extensions of the structure in order to ensure that soil–structure interaction is taken into account.

The properties of the soil should be established to a degree of accuracy that is sufficient in terms of the design assumptions made in order to achieve the required reliability. Upper and lower limits of estimated settlement, swelling or heaving, should be used to establish extreme effects on the structure.

The proper investigation of the subsoil conditions is thus essential. The nature and extent of such investigations will depend on the nature of the subsoil and the type of structure and its foundations and should preferably be planned in stages in conjunction with geotechnical experts in order to optimize the expenditure in terms of risk, as discussed in general terms in Chapter 3, Section 3.2. The details of the necessary procedures, exploratory fieldwork and laboratory testing are beyond the scope of this book, and reference should be made to suitable texts on the subject. The results of preliminary investigations may result in a revision of the planned investigations, or even a reconsideration of the type of foundations and even of the type of structure. Where short- or long-term settlements are predicted, it is essential that the structure be designed to accommodate these or that the type of foundations or its founding depth be determined so as to limit the movements to within the permissible range for the particular structure envisaged. The possibility of scour action around piers or abutments in structures spanning flowing water should be investigated and the design adapted accordingly. The banks of gorges or embankments may require protection, for which various techniques are available, or stabilization by retaining structures, or anchoring techniques (see Section 2.4.5).

2.4 Actions and Effects on Bridges

2.4.1 Definitions of Actions[2.1]

An action is an assembly of concentrated or distributed forces (*direct actions*), or imposed or constrained deformations (*indirect actions*), applied to a structure due to a single cause. An action is considered to be a single action if it is stochastically independent, in time and space, of any other assembly for forces, or imposed or constrained deformations, acting on the structure. Actions can be qualitatively classified according to their variation in time, or space, or according to their dynamic nature.

2.4.2 Classification of Actions[2.2]

It is convenient to subdivide direct actions into

(a) *principal actions*, which are gravitational forces;
(b) *supplementary actions*, which are usually applied separately in combination with associated principal actions on the basis of risk considerations.

Principal actions include:

(a) *Permanent and long-term actions*, such as
  (1) dead loads;
  (2) superimposed dead loads;
  (3) earth pressure due to retained fill;
  (4) water pressure of excluded or retained water.

(b) *Transient and variable actions*, such as primary live loads due to:
  (1) vehicle traffic loading;
  (2) railway loading;
  (3) footway loading;
  (4) cycle loading.

(c) *Short-term actions* such as erection loads. Dynamic or impact effects are usually included or allowed for by equivalent static impact factors.

Supplementary actions include

(a) *Transient secondary forces* due to primary live loads of highway traffic such as
   (1) centrifugal forces;
   (2) longitudinal braking and traction forces;
   (3) forces due to accidental skidding;
   (4) impact due to vehicle collision with bridge ballustrades or parapets;
   (5) impact due to vehicle collision with bridge supports;

and due to primary live loads of railway traffic such as
   (6) lurching effects which result from the temporary transfer of part of the live loading from one rail to the other;
   (7) nosing which allows for lateral loads applied by trains to the tracks;
   (8) centrifugal load effects on curved tracks;
   (9) longitudinal loads due to traction and application of brakes;
   (10) derailment loads.

(b) *Transient forces due to natural causes*, such as
   (1) wind action;
   (2) flood action;
   (3) earthquake action (seismic forces).

Indirect actions include imposed deformations or restraint actions of long-term effects due to

(a) creep and shrinkage of concrete;
(b) parasitic (hyperstatic) prestress and prestrain;
(c) differential settlement;

and of short-term effects due to

(d) temperature range;
(e) temperature gradient.

In limit-state codes of practice (see Chapter 3, Section 3.4) these various actions are specified as nominal forces in the absence of accurate characteristic values and are increased by partial load factors to derive the design loads. Various combinations of the actions are applied in which the partial load factors of the co-existent forces are varied so as to maintain a consistent level of probability. Reduced loads or the relieving effects of actions are taken into account (where applicable) with reduced partial load factors.

The responsible government authorities usually specify the actions (loads) that are to be applied in the design of bridges in their area of jurisdiction and require compliance with specific codes of practice covering the structural design. Only a few general comments will be made here.

Unfortunately, there is as yet little uniformity between the codes used in various countries. This is especially true of the design actions and various comparative analyses[2.3–2.5] of, for example, highway traffic loading as specified in countries such as the United Kingdom, France, Switzerland, the Federal Republic of Germany, Italy, the Netherlands, Belgium, the United States of America, Canada, Australia, South Africa and Japan, which show an astonishingly large variation in the type of loading models used and the magnitude of axle and total loadings. The order of these differences far exceeds real traffic differences. Various attempts made by members of the International Association for Bridge and Structural Engineering (IABSE) have not been successful in obtaining greater uniformity.[2.6] It would therefore appear that some countries are less conservative in terms of normal traffic, i.e. those vehicles that comply with the relevant regulations but then have to impose more severe limitations on abnormal loading.

In the case of Europe, the formation of the European Economic Community (EEC) and the opening of borders is naturally going to result in a more uniform approach in the future. It is apparent that increasing emphasis will be given to the preparation of Eurocodes and supporting European standards, published by the European Committee for Standardization (CEN) and based on international standards wherever possible, in replacement of existing national codes and standards.

2.4.3 Highway Traffic Loading

Traffic loading can conveniently be subdivided into three main classes:[2.7] normal traffic, abnormal vehicles and superloads. Normal (or standard) traffic consists of all vehicles with axle-load arrangements that comply with statutory loading regulations; abnormal vehicles exceed such limits and therefore require exemption permits; special abnormal vehicles, including superloads, are vehicles which because of size and/or mass require special permits and are usually required to travel under surveillance of an escort representing the local authority. Most codes make provision for normal traffic and some form of abnormal (or military) loading.

In some countries,[2.2,2.8–2.11] provision is also being made for superload vehicles which consist of slow-moving, multiwheeled trailers with controlled hydraulic suspension and steering (either drawn by haulers or self-propelled) with payload capacities of up to 400 tonnes and a gross mass of over 600 tonnes. These vehicles use special routes which have to be identified and upgraded where no existing roadway is available with sufficient load-bearing capacity and/or headroom clearance under overpass bridges. The by-passing of sub-strength bridges may be costly. Economic analyses indicate that the upgrading of new and even existing bridges, where feasible, can be justified on selected routes.

The loading models used for normal traffic vary considerably. Some authorities use groupings of wheel or axle loads, representing one or more actual vehicles, plus a uniformly distributed loading on the rest of the roadway length and width. Others apply formula loadings that are uniformly distributed but reduced in intensity for increased loaded lengths, in conjunction with one or more knife-edge loads or wheel loads which are applied according to specified rules in order to simulate approximately the effects of actual traffic. Some codes use the concept of lane loading, which may vary in intensity for adjacent lanes. These loadings either include impact effects, or otherwise require multiplication by an equivalent static impact factor. Various studies[2.12–2.16] have been made of statistical information about the distribution of vehicles along highways and in the different lanes. Insufficient statistical information is, however, at present available about traffic in general. It is not a simple random phenomenon, but is conditioned by the characteristics of a particular route, the type and density of traffic and such effects as the formation of a queue behind a heavy vehicle which tends to accumulate other heavy vehicles which cannot overtake as readily as lighter ones. Furthermore, traffic behaviour is subject to human response and direction, which may result in heavy closely-spaced vehicles in convoy. It can readily be shown that, except for the very small loaded lengths, the worst loading condition occurs under jam-packed (bumper-to-bumper) conditions caused by a traffic blockage and that the dispersion of traffic at speed, caused by the increased vehicular interspacing, more than offsets the effects of the dynamic impact of axle-loads.

It is clear that the distribution of the heaviest probable accumulations of jam-packed normal vehicles on a specific bridge will at some time in its design life tend towards a definite worst loading case for any element or part of the bridge. This usually results from a concentration of vehicles with maximum permissible axial loads (with some probably illegally overloaded) in close proximity due to the queueing effect. Such queues may occur simultaneously in adjacent lanes in one carriageway or in opposite carriageways. For design purposes it is required to know how many of these heavy vehicles could probably be involved and what the compositions and distributions of all these vehicles could be. It would depend on the characteristics enumerated above and could vary for different bridges. Any single formula loading would, in order to remain uncomplicated, have to represent a 'worst probable case'.

The above-mentioned probable arrangements are simulated in various ways in different codes. The British BS 5400:Part 2 (1978) formula HA loading allowed for 30 m of maximum uniformly distributed nominal loading of 30 kN/m of lane on two lanes with reductions on other lanes and a reducing loading expressed by an inverse exponential function, $W = 151(1/L)^{0.475}$ kN/m of lane, for loaded lengths in excess of 30 m up to 380 m, at which it becomes constant at 9 kN/m. Knife-edge loads of 120 kN/lane on two lanes with reductions for other lanes were applied in conjunction with the above loading. A single nominal wheel load, 100 kN, was applied separately.

This loading has, however, been superseded because of experiences of exceptional overloading due to abnormally long queues of heavy vehicles resulting from accidents or breakdowns causing fatigue failure of components on the Severn suspension bridge. The new uniformly distributed loading in association with a knife-edge load of 120 kN/lane consists of much higher intensities in the very short loaded-length range to the formula $260/L^{0.6}$ kN/m which is applicable for lengths up to 52.16 m, beyond which the formula $36/L^{0.1}$ kN/m applies. At 380 m the above loading is equal to 19.88 kN/m compared with the 9 kN/m of BS 5400:Part 2 (1978).

The effect of this very significant increase in loading in the short-span range must be assessed against that of the 1000 kN abnormal HB vehicle loading. There has however been almost a doubling-up of the effects on longer spans.

These developments demonstrate the problems of simulating traffic loading accurately for a range of bridges, and of anticipating future trends.

Accidental collision impact loading is usually specified in the form of equivalent static loads to be applied at specified levels against balustrades and piers. This is a useful way to ensure a minimum degree of robustness, but it does not give a true indication of the actual effect of impact due to vehicles at speed. The effect is dependent on the impulsive response of the balustrades or piers and the load-time characteristics of the colliding vehicle. A correct dynamic analysis is highly complex so that present designs are largely based on tests, but progress is being made with theoretical studies of concrete structures under impulsive loading.

2.4.4 Wind Action

Wind action and its effects on a bridge depend on its geographical location, the local topography, the roughness of the natural or built-up urban terrain which can have significant influence on the wind speed profile, the height of the relevant bridge member above ground, its direction relative to prevailing winds, and the horizontal and cross-sectional dimensions of the element under consideration. The maximum pressures or suctions are due to gusts that cause local and transient fluctuations about the mean values. The natural frequency of the bridge or section of bridge under consideration can also in large flexible structures have an influence on the wind action. The accurate determination of these effects is extremely complex. Reference should be made to suitable texts on the structure and aerodynamic behaviour of wind, its effects on and interaction with various types of structural elements and the aeroelastic response of structures thereto.[2.17–2.22]

Wind actions are variously specified by equivalent static, or semi-dynamic methods (see Chapter 4, Sections 4.4.4 and 4.6.2). In these methods, the dynamic pressure head acting on the relevant part of the structure is either determined in terms of the maximum probable mean hourly wind speed, or the maximum probable wind gust speed based on short period gusts of 3 to 10 s, in a specified period (usually the expected lifespan of the bridge, variously taken as up to 120 years).

Although some current codes of practice are based on the latter procedure, the maximum probable mean hourly wind speed is being accepted as the more reliable approach. The effects on relevant members of the structure are determined by the application of theoretical methods and formulae that have been developed as a result of research in the form of wind tunnel tests on scale models and recordings on prototypes. The formulae are evaluated by using various coefficients and factors which allow for:

(a) the degree of exposure (depending on the surface roughness of the terrain, the degree of shielding and the effective height of the relevant member);
(b) local acceleration of the wind due to funnelling effects in the case of bridges in valleys or over gorges or where the bridge is sited to the lee of a range of hills or on an escarpment;
(c) correlation of gusting effects dependent on the size and horizontal length or the height of the member to which the wind loading is being applied; and
(d) a drag coefficient which depends on the shape of the relevant part of the structure and the effects of frictional drag thereon.

The procedures for doing more accurate dynamic simulations have, in recent years, been greatly improved through the use of computers. In the case of very long, slender, wind-sensitive structures or members, reference should be made to suitable texts that describe methods of performing dynamic analyses based on the statistical approach in determining wind forces and the aeroelastic structural response, as well as the possibility of aerodynamic instability as a result of wind-excited oscillations (see also Chapter 3, Section 3.2.5, and Chapter 4, Sections

4.4.4 and 4.6.2). It may in such cases also be necessary to do wind tunnel tests, but the natural frequency of vibration of concrete bridges is usually of a sufficiently high order to require only one of the simplified methods specified in various codes. This is not necessarily true of the cables of cable-stayed concrete bridges, which have been known to develop excessive wind-excited oscillations.

The degree of sophistication of the analysis required for concrete bridges and construction components can usually be related to the probable maximum mean hourly wind speed appropriate to a return period equal to the expected lifespan of the bridge, the fundamental natural frequencies and the wind-loaded lengths of critical members.

The following rule can be used as a very rough guide only:

> A concrete structure or any concrete member of the structure is unlikely to be susceptible to the dynamic effects of wind action if $n_0 > 4v/L_w$ or $n_0 > 0.5$ Hz, where n_0 is the fundamental natural frequency and L_w the wind-loaded length respectively of the structure or member being considered and v is the mean hourly wind speed.

In the case of bridges or members where $n_0 L_w > 8v$ (approximately), nominal static horizontal forces equivalent to the effects of wind gust speeds expected in the area can be applied to the structure in accordance with procedures prescribed by local authorities. For cases where $n_0 L_w < 8v$ (approximately) an equivalent static wind force[2.2] should be applied; this is based on the dynamic pressure head acting on the projected solid area of the relevant part of the structure and takes into account the shape thereof and the effects of frictional drag thereon. The dynamic pressure head is proportional to the square of the maximum wind gust speed and depends on the density of the air (site altitude above sea-level and atmospheric pressure). Nominal values are used.

The maximum wind gust speed is a function of the mean hourly wind speed appropriate to a height above ground level of 10 m in open level country, and a 100-years return period, for which values in m/s can usually be obtained from a map of isotachs. These maps do not necessarily provide for localities where very severe localized winds may occur and which require special study. The above-mentioned mean hourly wind speeds must be modified by coefficients related to:

(a) the return period appropriate to the specific bridge, which would vary logarithmically from about 1.0 for highway bridges corresponding to 100 years, to 0.82 corresponding to 10 years;
(b) an exposure factor, which depends on the surface roughness of the terrain, the degree of shielding or exposure, and the effective height of the structure or member above ground level;
(c) a funnelling factor in valleys and gorges or where the bridge is

situated to the lee of a range of hills or an escarpment causing local acceleration of wind; and

(d) a gust factor dependent on the size and horizontal length or height of a member to which the wind loading is applied.

The above effects are given in differing format in various codes of practice with the necessary elaboration of treatment of relieving effects on parts of the structure and the forces acting on live load. Drag coefficients for various cross-sectional shapes of bridges and members are based on wind tunnel tests on equivalent models.

2.4.5 Flood Action

The maximum probable magnitude and nature of the flow of a river in flood depends on its geographical location, the local topography and the various meteorological factors or weather systems that generate rainfall in the catchment area. A bridge spanning a river and especially the filled approaches restrict the flow and may during flooding be subjected to severe hydrodynamic forces and scouring actions. The backing-up effect may flood surrounding areas. It is consequently very important to locate the bridge and the approach roads so that the total utility including the costs of the bridge structure as well as the roadway system are optimised in terms of all probable benefits and damages (see Chapter 3, Section 3.2.1 and Chapter 5, Section 5.2).

The procedures for designing bridges spanning waterways are normally prescribed by the governing authorities. These include methods of obtaining the necessary hydrological data in order to predict the probability and approximate magnitude of design floods and to assess the hydraulic behaviour of the flow through the substructure of the bridge in order to determine the probable hydrodynamic effects on the river morphology especially in the vicinity of the bridge and on the bridge substructure and foundations.[2.23–2.25] Faraday and Charlton[2.23] give a detailed flow diagram illustrating the relevant hydraulic factors in a typical procedure for design of bridges over rivers. It demonstrates the steps necessary to optimize the design. The objective is to determine a waterway opening and arrangement of piers and abutments that will result in backwater effects and general scour depths that are acceptable. Appropriate formulae are given to determine the hydrodynamic forces of drag and 'lift' (by analogy with the wing of an aircraft) that act horizontally on the piers. In a similar publication by the Roads and Transportation Association of Canada, Neill[2.25] also deals with problems related to tidal crossings.

The impact of debris or the piling up of brushwood against the piers and superstructure may magnify the effects. Where the deck is partially or totally submerged, buoyancy forces may reduce the stability of the structure. Where applicable, the forces generated by the interaction of ice and the bridge substructure should be assessed. The critical mode is most likely to be the impact of large sheets with piers or piles at ice break-up.[2.23]

The recurrence in recent years of serious damage to bridges in various countries, largely due to scour, but also due to excessive forces caused by floods greater than predicted, indicated the need for a reassessment of the design procedures and protective construction methods.[2.26,2.27]

Pier and abutment foundations need protection against scour by ensuring that the foundation depths are adequate and/or by applying a rip-rap apron. Where piles are used, the design should be based on the assumption that the piles are exposed down to the anticipated scour depth. Approximate formulae are available[2.23] for determining depths of scour adjacent to various shapes of piers in cohesionless as well as in cohesive soils. Unless the piles are very large, raking piles (with or without vertical piles), suitably arranged, provide an adequate solution. Suitable methods of protection of bank and slope revetments[2.24] as well as substructures include[2.23] deflecting groynes, guide banks or spur dykes.[2.24] Rip-rap, gabions, wire mesh mattresses filled with stone, sheetpiling, precast concrete blocks, *in situ* concrete or combinations thereof have been effectively used in their construction.

The need for reassessments applies equally to the interpretation of data used for flood prediction and methods of assessment of risk in order to ensure a sound basis for achieving an adequate reliability of design (see Chapter 3, Section 3.2). It has been general practice to base the design of bridge waterways on the effects of a flood with a return period that equals the anticipated useful lifespan of the bridge, usually assumed to be between 50 and 120 years but depending on the importance of the structure. Assuming the validity of the relationship between the severity of the event and the probability of its occurring during the design life as given by $p = 1(1-1/T)^L$ where p is the probability of an event having a return period of T years occurring at least once during the design life L, the probability of such natural peak events being exceeded at any time within the assumed period is approximately 64 per cent. The risk which is implied thereby would appear to be too high, especially in cases with high potential losses. The problem is exacerbated because of the sensitivity of bridge structures to the exponential increase of the effects of exceeding expected limits. It can consequently be shown that greater reliability would be achieved by basing the design on much longer recurrence intervals with load magnitudes which should not be less than a specific percentage of that due to a postulated maximum probable flood and which would then be associated with factors of safety much closer to unity (see Chapter 3, Section 3.2.5).

On account of the complexity of the problem, the procedures for predicting maximum probable floods are a combination of theoretical and empirical methods that depend on many variables and for many regions rely on limited statistical records covering periods much less than the design life. Although considerable research has been done the reliability of prediction is uncertain because of the nature of the problem, i.e. that very long-term peak floods can occur at any time.

The partial load factor for the ultimate limit state usually specified for flood action is of the order of 1.30. This factor does not allow for any substantial increase in the level of the predicted maximum flood or velocity of flow which would cause an exponential increase of the forces and moments acting on the structure. Any exceedance of the velocity of flow increases the load effect on the structure quadratically, and so does an increase in depth of flow increase the overturning effect. The effects of design floods as determined in current practice can therefore be perceived to be a probable maximum with the exceedance margins of velocity and depth of flow allowed for in a partial load factor of, say, 1.30 being less than 14 per cent taken singly, and even less when in combination.

In practice the strength and stability factors of safety and any redundancy, where present, would provide a reserve of resistance, but the reliability is uncertain, bearing in mind the unpredictability of the design flood levels. It has become evident in recent years that both currently-used methods of estimating probable maximum flood peaks, i.e. the statistical method and the deterministic method, are unreliable in certain regions if based on relatively short records in the range of 75 years, as are often only available.[2.28,2.29]

It would therefore seem that bridges should be designed against catastrophic collapse for extrapolated flood magnitudes with a return period of the order of 2000 years using a partial load factor approximating to unity. Alternatively, use should be made of the Francou–Rodier method[2.28] to determine regional maximum floods. The recommended design flood (RDF) with a very small exceedance probability could then be reduced by means of suitable reduction factors based on lesser risk implications where applicable in the case of particular bridges. Where adequate records are available, it would be prudent to apply both methods and determine the design flood by judgement based on 'notional' risk assessments.

It is clear that the procedures used in the past to design bridges against extreme floods need reconsideration. The methods used to evaluate the probable maximum flood during the lifetime of a bridge, usually assumed to be 100 years, have patently been inadequate.[2.28,2.29] Suggestions towards obtaining more realistic predictions have been made above. Alternatively, sensitivity studies of a bridge structure are necessary because of the above-mentioned exponential increase in the effects of the exceedance of design floods. The intensity of forces acting on bridge elements are proportional to the second power of velocity of flow, and overturning moments can therefore be proportional to the product of the velocity squared by the depth of flow squared, with a dramatic increase if the deck is partially or totally submerged.

Having determined a maximum probable flood as suggested and applying it with a load factor close to unity, the structure should be designed using the normal partial load factors for other actions and the normal strength and stability factors (see Chapter 3, Section 3.2). The risk of catastrophic collapse should, however, be minimized by applying specific design strategies, such as robust design incorporating redundancies where feasible. Alternative approaches, e.g. designing for overtopping of the bridge and/or approach fills and allowing for the washing-away of the latter to reduce the forces on the structure, should be assessed on the basis of risk analysis.

2.4.6 Earthquake Action

In countries where the probability of occurrence of earthquakes is significant, design for seismic effects is usually prescribed. On account of the historical record of the serious consequences of numerous earthquakes in specific zones, much research has been done in order to gain an understanding of the phenomenon and to facilitate the prediction of the maximum magnitudes of probable recurrences within the useful lifetime of bridges at specific locations. A large literature has developed and procedures for the simulation of earthquake effects and the response of structures thereto are available. Reference should be made to Chapter 15 of Ref. 2.30 by Fintel and Ghosh and to relevant codes and texts.[2.31–2.34]

Methods of simulation of earthquake actions on bridges and the structural response thereto are described in outline in Chapter 4, Sections 4.4.5 and 4.6.

Earthquake magnitudes are measured in terms of the Richter scale but for design purposes earthquakes are usually classified in terms of the Modified Mercalli (MM) scale (Table 4.2) giving the intensities at epicentres and the corresponding nominal ground accelerations. Approximate earthquake intensity zones are depicted on maps derived from the distribution of the expected intensity levels that have a specific (usually 90 per cent) probability of not being exceeded in a specific period (usually 100 years).

The structural response of a bridge depends not only on the seismic characteristics of any particular earthquake but also on the natural frequencies and damping ratios (Table 4.1) of the structural members and their configuration as well as on the soil-structure interaction (Table 4.3). The nature and location of joints and bearings can also have a marked effect on the structural response. The accurate analysis of earthquake effects can therefore be highly complex.

For zones with very low expected intensities a static method which represents the minimum requirements for any bridge structure, and which does not simulate the dynamic effects of an earthquake, may be used to ensure that the bridge has a nominal capacity of resistance against horizontal inertial forces of the order of 0.05 times the dead loads and superimposed dead loads of the structure applied at the centres of gravity of the masses and distributed accordingly.

For bridges varying from those with a low susceptibility to earthquake actions and situated in zones with a 90 per cent probability that Class viii seismic intensity, in accordance with the MM scale ($0.1g$ ground acceleration), will not be exceeded in 100 years, to those structures that are vulnerable and situated in zones in which Class vi ($0.03g$ ground acceleration) will not be exceeded, an approximate equivalent static method will usually be adequate. Structures with a low susceptibility to earthquake action are those with periods shorter than 0.05 s and larger than 3.0 s. Methods based on procedures such as those specified in the National Building Code of Canada (1980) are suitable.

For a range of bridges varying from those with a low susceptibility to earthquake actions and situated in zones with a 90 per cent probability that Class viii seismic intensity on the MM scale ($0.1g$) may be exceeded in 100 years, to those structures that are vulnerable and situated in zones in which Class vi may be exceeded, a quasi-dynamic simulation of the response of single-degree-of-freedom structures to the base excitation caused by earthquake is advisable. This method can be used as a manual method on small structures or as a computerized method on larger structures. In a multi-degree-of-freedom system that has independent or uncoupled modes of deformation, each mode responds to the base motion of excitation as an independent single-degree-of-freedom system. Generally, only the response in the fundamental or lowest natural response frequency is of interest to the designer.

The procedure of this method is briefly described in Chapter 4, Section 4.6.1.1. Adjustments for elastic-plastic behaviour are described in Section 4.6.1.2 and application to multi-degree-of-freedom structures in Section 4.6.1.3.

In the design of bridges against seismic effects it is important to avoid configurations that could result in a large concentration of reactive forces. The unavoidable elongated form of a bridge in plan is unfortunately an additional complication, as skewed transverse seismic waves will not act uniformly along the length.

The details of expansion and contraction joints should provide a sufficient gap between the concrete portions so as to prevent these from hammering against each other and the damaging of the joint details. The provision of bearings that allow a degree of lateral movement between the deck and short rigid columns may significantly reduce the reactive forces thereon by transference to more flexible columns provided that ultimate stability is maintained. The Bloukrans bridge on which this method was adopted in the design of the spandrel columns is illustrated in Figs 1.11 and 5.43. Laminated lead–rubber elastomeric bearings furthermore increase the amount of damping because of the hysteretic properties of the inelastic deformations.

Although not commonly used in bridges, devices that isolate the bridge at its bases from intense earthquake motions deserve consideration. There are basically two requirements of such a device, viz. horizontal flexibility and energy-absorbing capability. These devices are used on building structures that have a stiff superstructure and concentrate most of the horizontal deformation to the flexible mountings that support the structure. Various systems applicable to buildings have been investigated,[2.35] and application to bridges with stiff piers is relevant. Bearings that make provision for longitudinal movements due to shrinkage and temperature differentials can also be provided with the above-mentioned characteristics.

The performance criteria at the ultimate limit states are important and the safety against the maximum expected earthquake must be assured. Inelasticity should be confined to bracing beams with limits on deformations to ensure stability of columns. For the structure effectively to resist and absorb the seismic effects, sufficient ductility should be provided in those bracing beams that are designed to achieve prior inelasticity.

Redundancy has a significant effect on improving the ultimate resistance of a bridge structure against seismic effects (see Chapter 3, Section 3.2.6). However, the sequence of development of failure mechanisms is important to prevent prior collapse due to gravitational forces. The failure of bracing beams should precede that of columns. In the case of very tall columns excessive drift may induce overstressing due to the $P\Delta$ effect. The proportioning and detailing of members and joints and the anchorage of reinforcement is critically important. The structural units should be as continuous and monolithic as possible and various elements that may be required to interact effectively should be very adequately joined together to transfer the effects of the seismic-induced forces.

2.4.7 Restraint Actions

The effects considered above all result from body forces or externally applied actions. Restraint actions, on the contrary, are due to induced dimensional changes or deformations which generate reactions in constrained structural members forming part of a statically indeterminate system. Creep and shrinkage of concrete and differential settlements are examples of restraint actions. The primary applied forces due to prestressing are usually treated as load-resisting forces (see Chapter 4, Section 4.4.2) but the secondary or parasitic forces (also called hyperstatic forces) in statically indeterminate beams can also be classified as restraint actions. The stress effects caused by these induced deformations depend on the stiffness of the relevant structural members and the rigidity of the restraints. On account of the non-linear behaviour of concrete as the ultimate limit state is approached and the resultant reduction in the effective stiffness of the relevant members, the restraint effects constitute a larger portion of the stress effects for a serviceability limit state condition than for an ultimate limit state condition. As most codes recommend that analysis should in general be performed for service actions imposed on structures consisting of members whose properties are evaluated on the basis of elastic material behaviour, an over-estimation of the effects of restraint action may be made in the case of certain ultimate combinations. It would therefore be reasonable to reduce the effects of restraint actions by a suitable adjustment of the relevant partial coefficients where the failure mechanism induces relief from the effects of the above-mentioned restraints.

2.5 Factors Affecting the Choice of Configuration, Materials and Methods of Construction

Almost every major bridge that is designed seems to be unique. Except for a degree of standardization that has at times been achieved on certain projects for bridges in the short-span range, for example freeway structures, bridge designers tend to produce results that reflect their personal preferences. Progress in knowledge, improved quality of materials and advanced methods of analysis and construction do naturally lead to a greater scope in possible alternatives, but personal ambition to innovate is probably also a strong motivating factor. This in itself is not necessarily a bad thing, provided it does not lead to innovation for the sake of innovation and does not disregard the economic implications. It would appear that almost every useful type of configuration that is conceivable has been utilized to date, but almost endless variations or combinations thereof are certainly still available to be exploited.

Experience has shown that certain configurations (see Chapter 1, Section 1.3) are most suitable under specific circumstances that can be related to the functional requirements, the site conditions, the magnitude of span(s) required and the materials and the methods of construction used. These factors are all interrelated so that there are usually several alternative solutions that may be competitive. *The objective of the designer is therefore to find the combination that gives an optimum solution in terms of the functional requirements and cost, as well as the broader issues referred to in Chapter 1, Section 1.5, and based on the design philosophy described in Chapter 3 or such other as is prescribed.*

The procedure to be adopted in order to achieve this result is typical of most design procedures (see Fig. 3.1) which require the development of several conceptual designs incorporating all feasible alternatives and by the evaluation and comparison of these, reducing the number of possible solutions in stages by the process of elimination (see also Chapter 5, Section 5.9). Past experience may enable a designer to commence with a greatly reduced number of alternatives. It is not easy to describe the conceptual process which, unless it amounts to merely adapting existing designs, requires innovation, the nature of which is briefly discussed in Chapter 1, Section 1.6. As there are so many alternatives and details that are relevant to this discussion, it will only be feasible to discuss the more important factors in broad outline and to give general guidelines.

The interrelationships referred to above are important in the selection of the exact location and it may be necessary to develop conceptual designs at alternative sites. The practical procedures for site investigation and selection are described in Chapter 5, Section 5.2. The practical details and construction methods referred to below are described in Chapter 5.

The profile and size of the space that must be spanned, together with the nature of the founding conditions, will usually narrow down the selection of the configuration considerably. In the span range beyond approximately 350 m, the dead load is a governing factor so that only steel cable-suspended bridges have been built to date although concrete towers and substructures were used for the Humber suspension bridge, with the longest span in the world of 1410 m (4626 ft). Cable-stayed bridges with steel decks are at present competitive in the range of 100–500 m (330–1650 ft).

In the range of 100–350 m, cable-stayed bridges with concrete decks are able to overcome the dead load disadvantage where the material and construction costs are lower. New developments will in all probability increase the range considerably (see Chapter 5, Section 5.7). Suspension bridges with draped cables and suspension hangers, apart from being the most competitive solution in the immediate past for spans in excess of 500 m, have the added advantage that centering is not necessary for the deck construction; this makes them eminently suitable for large spans over wide rivers or estuaries where the deck sections can be floated and hoisted into position to assemble the deck in a predetermined sequence. Cable-stayed bridges have a similar advantage and are also very suitable for construction over deep gorges, the deck normally being constructed by the suspended cantilever method with the cable-stays being applied after completion of the related segment of deck.

Concrete arches over gorges appear to be competitive up to spans approaching 400 m if the suspended cantilever method of construction is used, as described in Chapter 5, Section 5.6.

In the medium-span range, concrete girder bridges with a wide range of cross-sections are suitable for construction by the cantilever method (with or without temporary suspension) in which travelling falsework carriages are used. A wide range of specialized travelling steel girders that span the piers are available to support falsework for *in situ* casting of concrete as well as support for precast sections placed by cranes or hoisted into position. The systems are advanced ahead of the concrete superstructure and eliminate support by centering except perhaps at midspans, say, of larger spans. The use of these and other systems, described in greater detail in Chapter 5, Section 5.3, in preference to centering, will depend largely on the height of the superstructure above ground, the total length of the bridge (which determines the number of repetitive uses), and the availability or cost of the specialized girders.

The risks involved in constructing any particular bridge may play a significant part in the assessment of the cost of construction and should therefore be minimized.

The factors that determine the choice of the materials of construction are closely related to most of those factors already discussed in Chapter 1, Section 1.5, and in this section. Apart from the structural properties, special reference must be made to durability (see also Section 2.6, Chapter 6 and Chapter 4 of Ref. 2.30) in the particular circumstances of exposure such as the possibility of weathering, i.e. corrosion of steel, deterioration, discoloration, cracking and spalling of concrete, as well as fatigue under repetitive loading, any of which factors may not only have a significant effect on maintenance costs, but may disfigure or shorten the useful life of the bridge. The surface texture of the material and the variety of forms or shapes that can be constructed are very significant from an aesthetic point of view. The suitability of concrete in comparison with other materials is discussed in Section 2.6.

In all these considerations, the location and accessibility of the site, the availability of materials, the scale of operations and the time factor are very important. The contractor's capital resources and his general turnover of work of a similar nature will determine the scale of economic operations, namely how much he can invest in specialized temporary works, machinery and equipment in order to expedite the completion of the permanent structure. It is thus necessary to know what the capabilities and resources of available contractors are as this may have a crucial influence on the configuration of the bridge and the methods of construction. It is accordingly essential to take as many of these factors as possible into account when doing the conceptual design.

Various authors of papers and textbooks have prepared graphs comparing various types of bridge over the total practical span ranges. With the abscissa representing span length, the ordinate values are either the cost in a specific currency at a specific date, or the weight of steel for steel structures, or the volume of concrete for concrete structures, with percentages of reinforcement sometimes indicated. The cost may be for the superstructure only or may also include the substructure and the foundations. These graphs have limited usefulness, however, in giving a very approximate indication of relative costs. Practical experience has shown that if the optimum solution is to be found, every project requires a separate analysis of all feasible alternatives, except that previous experience under the particular circumstances can reduce the initial work drastically. The factors that influence costs as described previously are many and varied, and change with time. They may also differ substantially between countries or even districts because of local industry and vested interests.

Figure 3.1 is a greatly simplified abstraction of the design process, but demonstrates the method quite clearly. A large number of additional cycles of analysis and feedback are required to optimize the details.

2.6 The Suitability of Concrete as a Bridge Material

The properties and many uses and advantages of concrete in its various forms are covered in a comprehensive literature based on an on-going research activity aimed at improving all aspects thereof.[2.30] This includes the ingredient materials of aggregate, sand and cement with the ever-increasing variety of admixtures to improve the strength, durability and economy. Great advances are also being made in research into the microstructure of cement, in order to achieve a more basic understanding of its properties and behaviour. Here we are benefiting from the developments of improved tools in other disciplines such as secondary ion mass spectrometry, analytical electron microscopy, and X-ray fluorescence analysis. Although the phenomenological approach as widely used in the past will remain a very necessary method, more fundamental research presents promising prospects.

The improvement of concrete by the development of blended cements and admixtures has been considerable and experimentation in this direction is ongoing, the objective being to improve the properties without any substantial increase in cost. Progress is being made with high-strength concretes, fibre-reinforced concretes, structural light-weight concretes, and polymer-impregnated concretes for special purposes. In the more pragmatic area of concrete mix-design there is still room for improvement, and the growing shortage of suitable aggregates is cause for concern in some locations.

Concrete is especially suitable as a bridge material, as has been proved by the ever-increasing use of it in most parts of the world relative to other structural materials. The reasons therefore are mainly functional and economic, but the aesthetic advantages are considerable. It can be readily moulded into almost any form that is desired and, if constructed with care, has a texture which blends well with the natural environment. The basic requirements to satisfy aesthetic standards, as described in Chapter 1, Section 1.4, can, in general, more readily be satisfied in concrete. This applies to almost all configurations in the short- and medium-span range and for arches up to 400 m span. However, even concrete cable-stayed decks can be very pleasing to the eye as they blend in well with the concrete towers and substructures while contrasting effectively with steel cables. On the other hand, the Humber suspension bridge is a good example where a steel-decked suspension bridge has been very satisfactorily combined with concrete substructures and towers.

Although it is a brittle material, the properties can be transformed from that of ordinary reinforced concrete, through partial prestressing to full prestressing, to have the required structural properties at the lowest cost. Prestressing is usually applied only in one or two directions corresponding approximately with principal tensile stresses, with ordinary reinforcement taking transverse, shear, torsional, secondary and bursting stresses. In the case of structural components involving high tensile stresses in several or varying directions, prestressing has, however, been applied in multiple directions.

Concrete as a material can thus be moulded into virtually any shape that is required; it can be coloured by using suitable admixtures and it can be given various surface textures for aesthetic effects. By the use of suitable reinforcement, it can be given triaxial structural properties to suit the specific requirements (see Chapter 10, by Hobbs, of Ref. 2.30). It can furthermore be cast *in situ*, precast in a factory or yard and transported to the site and placed on the works in a size and form to suit the circumstances and the contractor's resources. It can be combined with structural steel to give a composite construction (see Chapter 17, by Sabnis, of Ref. 2.30) in which form it very suitably makes the deck of a steel girder bridge.

In regions far from steel-producing industrial areas and where concrete aggregates and cement are readily available, it has the advantage of a big saving in transportation costs compared with structural steel.

Although concrete may suffer various forms of deterioration as described in Chapter 6, Section 6.2, and more fully in Chapter 4, by Campbell-Allen and Roper, of Ref. 2.30, the record of durability for well-designed and well-constructed concrete bridges is good and could, with greater care to detail and regular site inspections with well-managed maintenance programmes, be even better. Especially in areas with very severe climates and aggressive environments, great care is required to ensure a high quality of dense concrete with adequate cover to the steel reinforcement. It is essential to avoid recesses that might accumulate soil and moisture which could more readily lead to corrosion of the reinforcement if the concrete has areas of lower density or large cracks in the vicinity. Although cracks cannot always be avoided, good design and detailing can minimize their number and size. Shrinkage and temperature effects are major causes of cracks and full account should be taken of their effects, with the provision of suitable reinforcement or movement joints where necessary.

Concrete structures have a further advantage in that the inherent high dead-load-to-live-load ratios make them less prone to fatigue effects.

2.7 Properties of Reinforced/Prestressed Concrete as a Material of Construction

The structural properties of concrete as seen by the designer are described in Chapters 3, 4, 10, 11, 12, 16, 19 and 20 of Ref. 2.30. Reference should be made by the designer to the codes of practice that are applicable to the project he is involved in for the basic assumptions and definitions that prescribe the material properties and design methods to be used. Where outdated codes are still in use, it is recommended that safety aspects be checked in terms of the latest proposals contained in the CEB–FIP (1978) model code* for concrete structures as contained in Volume II of Ref. 2.1, pp 53–78 (including the notes), or any such other practical code that is based on the principles contained therein (see Chapter 3, Section 3.4, in this book and Chapter 40, by Rygol, of Ref. 2.30).

References

2.1 Comité Euro-International du Béton (CEB) 1978 *International System of Unified Standard Codes of Practice for Structures* vol I: *Common Unified Rules for Different Types of Construction and Material* 123 pp, and vol II: *CEB–FIP Model Code for Concrete Structures* Bulletin d'information, N. 124/125–E, 348 pp

2.2 National Institute for Transport and Road Research 1981 *Technical Methods for Highways: Code of Practice for the Design of Highway Bridges and Culverts in South Africa* TMH 7 — Parts 1 and 2, Pretoria, 100 pp

* A first draft of a very much improved *CEB–FIP Model Code 1990* (14 chapters and appendices) was published by CEB dated March 1990 for approval at the 27th Plenary Session of CEB in September 1990.

2.3 Seni A 1969 Comparison of live loads used in highway bridge design in North America with those used in Western Europe *Symposium on Concrete Bridge Design* American Concrete Institute, SP-26, vol 1, pp 1–34

2.4 Rajagopalan K S 1969 Comparison of loads around the world for the design of highway bridges *Symposium on Concrete Bridge Design* American Concrete Institute, SP-26, vol 1, pp 35–48

2.5 Liebenberg A C Mar 1974 *Report to the National Transport Commission of the Republic of South Africa on Proposals for a Uniform Specification of Live Loading due to Traffic on National Road Bridges* Liebenberg & Stander, Cape Town, pp 9–217

2.6 International Association for Bridge and Structural Engineering, British Group 1975 *Bridge Loading Colloquium* Cambridge, UK, April 1975, International Association for Britdge and Structural Engineering, British Group (unpublished, but available from IABSE)

2.7 Liebenberg A C 1975 The simulation of normal traffic loading on highway bridges *Bridge Loading Colloquium* Cambridge, UK, April 1975, International Association for Bridge and Structural Engineerings, British Group (unpublished, but available from IABSE)

2.8 Basler K 1967 Belastungsvorschriften für Brücken der Schwertransporten-strassen *Schweiz. Bauztng* **85**(20): 352–6

2.9 VSS-Arbeitsgruppe 13 1967 *Versorgungsnetz. Bericht an die kantonalen Baudirektoren zu technischen Lösungen* 6 Dez 1967, Berne, Switzerland

2.10 SURCH 71:1, Service d'Etudes Techniques des Routes et Autoroutes (SETRA) 1972 *Application des Nouvelles Charges Réglementaires des Ponts Routes — Notice Générale* DOA B, June 1972, 60 pp

2.11 Pennels E 1978 *Concrete Bridge Designer's Manual* Cement and Concrete Association, London, 164 pp

2.12 Ivy R J, Mitchell S, Raab N C, Richey V J, Scheffey C F Jun 1953 Live loading for longspan highway bridges *Am. Soc. Civ. Eng. Trans.* paper no 3708, separate no 198, pp 981–1003

2.13 Mitchell S, Borrmann G F Jul 1957 Vehicle loads and highway bridge design *J. Struct. Div. Am. Soc. Civ. Eng.* **83** (ST4): 1–21

2.14 Lynch R L Feb 1968 *Research of Traffic Loads on Bridges* Kentucky Dept of Highways, Division of Research, Lexington

2.15 Rey E 1970 Adjunkt Eidg. Amt für Strassen- und Flussbau, Bern: Strasse und Verkehr 7/70. Die Verkehrslasten der Strassenbrücken mit Normal-belastung *Strasse und Verkehr* no 7, 6 July 1970

2.16 Buckland P G, McBryde J P, Navin F P D, Zidek J V 1978 Traffic loading of longspan bridges (TRR 655) *Bridge Engineering* **2**: 146–54

2.17 Davenport A G Jun 1967 Gust loading factors *J. Struct. Am. Soc. Civ. Eng.* **93** (ST3): 11–34

2.18 Davenport A G Nov 1962 The response of slender, line-like structures to a gusty wind *Proc. Inst. Civ. Eng.* **23**: 389–408

2.19 The modern design of wind-sensitive structures, *Proc. Seminar of Institution of Civil Engineers* London, 18 June 1970, Construction Industry Research and Information Association, London, 1971, 139 pp

2.20 Hay J S 1980 *An Introduction to Some Current Theories on the Aerodynamic Behaviour of Bridges* Transport and Road Research Laboratory, Dept of Environment, Dept of Transport, London, supplementary report 542, 50 pp

2.21 Simiu E, Scanlan R H 1978 *Wind Effects on Structures: an Introduction to Wind Engineering* Wiley, New York, chapter 6

2.22 Biétry J, Sacré C, Simui E 1978 Mean wind profiles and change of terrain roughness *J. Struct. Div. Am. Soc. Civ. Eng* **104**(10): 1585–93

2.23 Faraday R V, Charlton F G 1983 *Hydraulic Factors in Bridge Design* Hydraulics Research, Wallingford, UK

2.24 Bradley J N 1970 *Hydraulics of Bridge Waterways* 2nd edn, Bureau of Public Roads, Hydraulic Design Series no 1, US Govt Printing Office, Washington DC

2.25 Neill C R (ed) 1973 *Guide to Bridge Hydraulics* Roads and Transportation Association of Canada, University of Toronto Press, Toronto

2.26 *Proceedings of Conference on Floods in Perspective* 1988, Div. of Hydr. and Water Eng. of the South African Inst. of Civil Engrs. 1988 CSIR Conf. Centre, Pretoria

2.27 Liebenberg A C Dec 1988 Risk analysis of bridge structures over rivers in South Africa *The Civil Engineer in South Africa* 551–554, 559

2.28 Kovãcs Z P 1981 *Maximum Flood Peak Discharges in South Africa: An Empirical Approach* Technical Report TR 105, Directorate of Water Affairs, Pretoria

2.29 Kovãcs Z P 1988 Preliminary hydrological assessment of Natal floods *The Civil Engineer in South Africa* 30(1): 7–13

2.30 Kong F K, Evans R H, Cohen E, Roll F (eds) 1983 *Handbook of Structural Concrete*, Pitman, London

2.31 Blume, J A, Newmark N M, Corning L H 1961 *Design of Multistorey Reinforced Concrete Buildings for Earthquake Motions* Portland Cement Association, Stokie, 318 pp

2.32 Newmark N M, Blume J A, Kapur K K Nov 1973 Seismic design spectra for nuclear power plants *J. Power Div. Am. Soc. Civ. Eng.* **99**: 287–303

2.33 Newmark N M, Rosenblueth E 1971 *Fundamentals of Earthquake Engineering* Prentice-Hall, Englewood Cliffs, NJ, 640 pp

2.34 Associate Committee on the National Building Code *National Building Code of Canada* NRCC 17303 and supplement NRCC 17724, National Research Council of Canada, Ottawa, 1980, 547 and 293 pp respectively

2.35 Lin B C, Tadjbakhsh I G, Papageorgiou A S, Ahmadi G 1990 Performance of earthquake isolation systems, *J. Engineering Mechanics, ASCE* **116**(2): 446–61

3 Design Philosophy

3.1 General

The very notion of structural design, even before it was properly understood, has always implied that safety was of prime concern. The concept of safety has developed from intuitive understanding to the modern philosophy of structural reliability,[3.1] which represents a significant advance in putting structural engineering on a more rational basis. Progress has not always been rapid, however, and misconceptions are common even today. Yet the lack of proper understanding of the functional behaviour of structures under imposed loads or influences does not, in general, appear to have greatly impeded engineers in the development of new and great structures to meet the ever-increasing demands of society. The remarkable enterprise and courage of engineers are demonstrated by the history of the development of bridges as related in Chapter 1, Section 1.2. They had to rely almost entirely on empirical knowledge of structural design prior to the second half of the 19th century; but these early designers did not always succeed at the first attempt, and many failures were experienced.

The history of the development of suspension bridges and the systematic study of wind forces, initiated by the Tay Bridge disaster in 1879, is well documented. In spite of this work, the spectacular failure of the Tacoma Narrows Bridge occurred because of lack of understanding of the nature of the torsional oscillations induced by wind action. The extensive research which was initiated as a result of this failure accomplished the dual purpose of determining the mechanism of the wind action which caused the failure and ensuring the aerodynamic stability of the proposed design for the new bridge. These and other failures illustrate man's readiness, even in relatively recent times, to undertake the design and construction of large engineering structures beyond his experience and full understanding. Over-design by the application of factors of ignorance could not guard against the effects of critical conceptual errors.

The development of modern high-strength steels which do not have a corresponding increase in fatigue properties and which are subject to the added risk of brittle failure, and the tendency to use very high-strength concretes, coupled with the greatly enhanced ability of the modern engineer to incorporate every part of the potential structural capacity in the strength analysis — thereby eliminating previously unaccounted for reserves — have made a sound understanding of structural behaviour and the nature of the applied forces imperative.

In spite of the accomplishment of major structural engineering feats in the past, the concepts of safety and risk have not until recently been very well understood. From an initial intuitive understanding based mainly on experience gained from studying failures, a misguided notion of the safety of structures was unfortunately fostered by the comprehensive deterministic design procedures that were developed since the theory of structural analysis was placed on a sound footing by men such as Navier. It was generally believed that the factors of safety which were applied to breaking or yield stresses of materials, in order to determine the safe working loads, were an almost absolute measure of safety, dependent only on the accuracy of linear structural theory and the material parameters. Furthermore, most major failures resulted from gross or conceptual errors so that the shortcomings of these deterministic procedures were not self-evident.

In spite of the development of wide-ranging research and testing techniques, the working stress method was until recently accepted as a satisfactory design method. Many practising engineers failed to realize that it did not result in consistent levels of safety.

The introduction of ultimate load checks based on an approximate theoretical simulation of the behaviour of structures at failure, although still deterministic, greatly improved the situation, but was not generally applied, although the application of a degree of redistribution of bending moments in the working stress methods became common practice. However, there had been an awareness of these inconsistencies by men such as Professor A. Pugsley,[3.2] who suggested the application of ultimate load methods with multiple load factors for various types of loading. Subsequently it was realized that an even greater multiplicity of factors needed consideration, such as the correctness of the basic assumptions of the theoretical methods of analysis, the accuracy of the calculations, and the degree of control of material properties and dimensional accuracy during fabrication and construction, as well as the seriousness and economic consequences of failure. In the asessment of collapse the use of statistics in defining loads and material properties was advocated.

3.2 Structural Reliability and Utility

Further attempts to rationalize the situation on theoretical lines led to

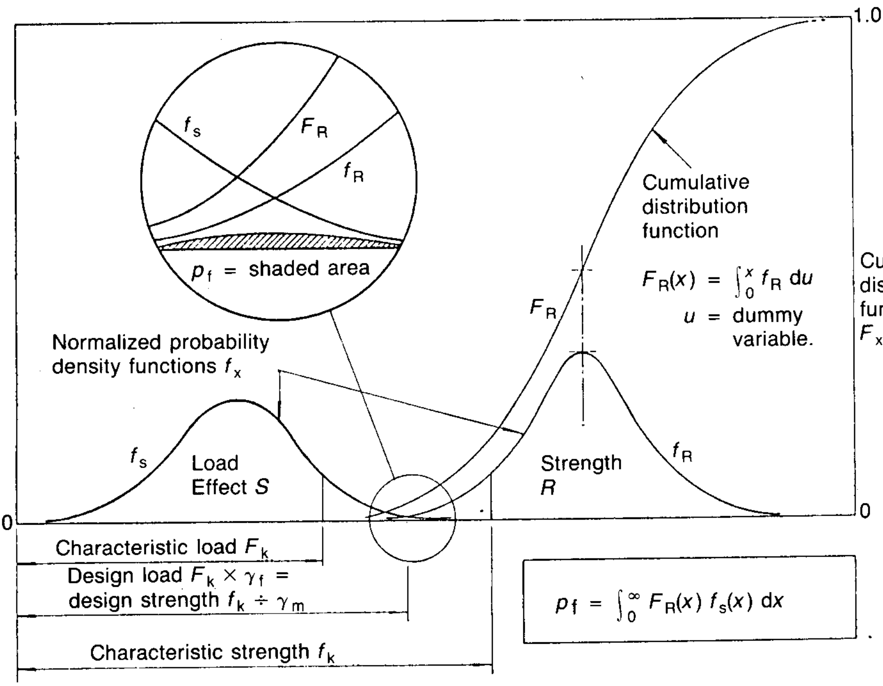


Figure 3.1 Probability of failure p_f in terms of classical probability theory

the classical theory of structural reliability, which can be defined as the 'measured chance' that a structure will support the loads to which it is subjected. Theory of structural reliability and the concepts of 'risk of failure' and structural 'utility', as developed by Freudenthal[3.3] and others, attempt to take into account the many uncertainties in the assumed applied forces, the properties of the materials and the construction techniques being used, by quantifying the relevant values in terms of statistical concepts and predicting the expected shortest operational life of an engineering structure by the application of probability theory. In classical reliability theory, the probability of survival or reliability is $p_s = 1 - p_f$, where p_f (the probability of failure) $= p(R < S) = \int_0^\infty F_R(x) f_s(x) \mathrm{d}x$, where F_R and f_s are respectively the probability distribution and density functions of R and S, the structural capacity and load effect respectively (see Fig. 3.1).

On the basis of this idealized theory, greater clarity of the fundamental principles of structural engineering has been achieved, but although this development has in a qualitative sense been a major breakthrough, it was not of immediate practical significance because of the difficulties in accurately quantifying probability levels due to lack of knowledge of the distributions of the relevant probability density functions.

The extended reliability concept by Ang[3.4] is an improvement on the above formulation as it eliminates the sensitivity of the semi-probabilistic methods to the shapes of the tails of the probabilistic distributions by defining failure as the probability that $R < NS$, where N is a 'judgement factor' and is necessarily greater than 1.0 to take account of unknown uncertainties.

The principles of reliability analysis, related more directly to safety, have been well documented and further research is an ongoing activity. The last two decades have seen the growth of a very substantial literature on the theory of structural reliability based on the statistical mean values and coefficients of variation, which are more readily determinable than the extreme values of the above-mentioned probability density functions that are necessary for the determination of the probabilities of failure in classical theory.

The first-order second-moment reliability analysis proposed by Cornell,[3.5,3.6] and further developed by Lind and others, is generally considered to be an improvement on the above proposals. Cornell's format provides a method based on distribution-free assessments and, within this simplified framework, approximately consistent reliabilities can be obtained. The code format used is to express the reliability of the structure (or member) in terms of the safety margin between resistance and applied force, i.e. $M = R-S$, the mean (expectation) and variance of which are $m_M = m_R - m_S$ and $\sigma_M^2 = \sigma_R^2 + \sigma_S^2$, respectively.

The reliability in this context is defined as the probability that M exceeds zero and is measured by the number of standard deviations σ_M by which the mean m_M exceeds zero (see Fig. 3.2). This number, β, given by

$$\beta = \frac{m_M}{\sigma_M} = \frac{m_R - m_S}{\sqrt{\sigma_R^2 + \sigma_S^2}}$$

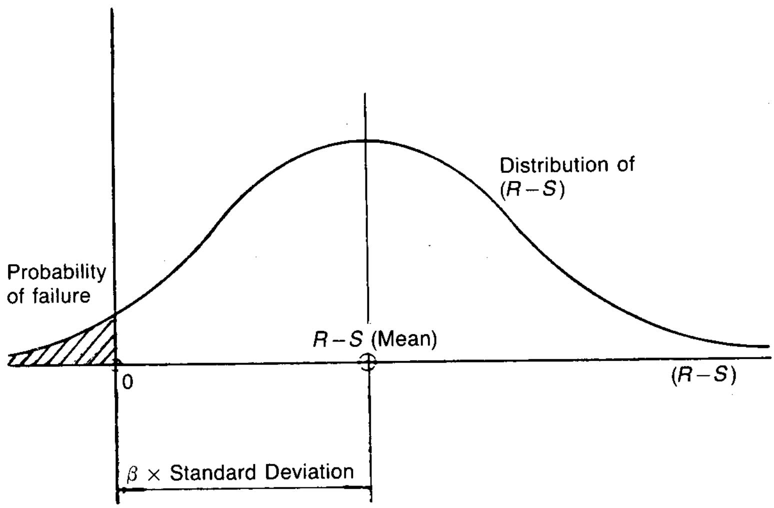


Figure 3.2 Definition of safety index β

is called the safety or reliability index. If $F = m_R/m_S$ is the central safety factor, it follows that

$$\beta = \frac{(F-1)}{\sqrt{F^2V_R^2+V_S^2}} \qquad [3.1]$$

where V_R and V_S are the respective coefficients of variation, so that F can be obtained from β, V_R and V_S.

Studies based on current reinforced concrete design practice in various countries indicate that target values of β chosen for the purpose of deriving partial load factors vary, but generally are of the following order:[3.7]

Ductile, gradual modes of failure: $\beta = 3.0$
Brittle, sudden modes of failure: $\beta = 4.0$

Partial factors so derived should be multiplied by an importance factor of up to 1.2 depending on the consequences of failure, or reduced by 0.9 where the consequences are not serious.

An estimate of the level of reliability associated with a particular value of β (or the complementary probability of failure or unserviceability) may be obtained from

$$\text{Reliability} = 1-\Phi\,(-\beta) \qquad [3.2]$$

where Φ represents the normal distribution function. This equation is only exact if the equations governing the behaviour of the structure are linear and the variables are independent and normally distributed.

Since the early work of Cornell, there has been considerable progress in the development of these methods, known as Level 2 methods. Level 2 is a probabilistic design process used principally in assessing appropriate values for the partial safety factors in the Level 1 method; thus it is intended primarily as a tool for code-drafting committees. The Level 1 method is the basis of various modern codes using the limit state design method. It is not possible to deal adequately with reliability analysis in this book. For an excellent summary the reader is referred to Esteva[3.8] and, for fuller details, to the CIRIA Report 63 of July 1977.[3.9]

Ultimately, the aim is to achieve optimal structural reliability by maximizing effectiveness expressed in terms of an objective function (utility function linear with money).

3.2.1 Structural Utility Function

$$\begin{aligned}\text{Structural utility} &= B-C_t \\ &= B-C_p-E_d \\ &= B-(C_i+C_m)-\Sigma(S_m \times p_m)-\Sigma(U_n \times p_n)\end{aligned} \qquad [3.3]$$

where

B = expected present value of the overall benefits derived from the existence of the structure (positive utility)
C_t = total capitalized costs (negative utility)
= (C_p+E_d) = loss function
C_p = capitalized prime costs = (C_i+C_m)
C_i = initial cost
C_m = capitalized normal maintenance costs
E_d = expectation of damages
S_m = capitalized cost of damage or loss due to non-compliance with serviceability criteria
p_m = probability of exceeding a serviceability limit state
$\Sigma p_m S_m$ = risk of exceeding a serviceability limit state
U_n = capitalized cost of reaching an ultimate limit state
p_n = probability of reaching an ultimate limit state
$\Sigma p_n U_n$ = risk of reaching an ultimate limit state

The generalized reliability can be defined as

$$R = \frac{C_p}{C_t} = \frac{C_t - E_d}{C_t} = 1 - \frac{E_d}{C_t} \qquad [3.4]$$

The various values that make up an important part of the terms denoted by B, S_m and U_n are, however, at present irreducible to a form that can be accurately quantified in practice. These concern human life and various subjective values that fall broadly in the domain of the aesthetic. Whether the 'economic probability' basis of the above comprehensive objective function can in practice be developed into a form that will be quantitatively meaningful is debatable in the light of the subjective content of some of the judgements that are an essential part of the process of assessing the relevant factors. In this connection, Bayesian analysis[3.5,3.10] promises to become a useful tool for the rationalization of the decision-making processes that depend on both subjective and objective information. There are also legal and ethical implications arising out of design procedures based on those concepts that need careful consideration and the fact that a structure is at risk during its planned lifetime has yet to be accepted by many controlling authorities and clients and even by the design profession as a whole. Despite these seemingly unsurmountable problems, this comprehensive approach is nevertheless a sound and necessary formulation and it is important to go through the exercise of doing such evaluations, even if only on a qualitative basis.

A useful result can be derived from a simplified version of the utility function applicable to a single member–single load case as follows. The probability of ultimate failure can be expressed as a decreasing function of capitalized prime cost or $p_n(C_p)$. It can be shown that the cost of failure, U_n, is usually only weakly dependent on C_p as the consequential losses can be far more serious and, in the range of high reliability, only minor variations in C_p are required to ensure significant increases in reliability. U_n can thus be considered to be constant. According to Lind and Davenport,[3.11] the probability of failure may be written as

$$p_n = \exp[-f(C_p)] = \exp[-f(C_p(F))] = \exp[-g(F)]$$

where $g(F)$ is a function that depends on the distribution of the safety factor F.

This function, if developed in terms of several mathematical distributions that arise out of various assumptions regarding the distribution of strength and resistance (normal, lognormal, extreme or Weibull type distributions), clearly illustrates the sensitivity of p_n to assumed distributions. More important is the fact that $\ln(p_n)$ is nearly linear as a function of the safety factor so that $p_n \cong b \exp[-C_p/c]$, where b is a constant and c is the attenuation cost, i.e. the cost of reducing the probability of failure by a factor of e($\cong$ 2.7).

We can therefore express the total cost as

$$\begin{aligned} C_t &= C_p + U_n p_n \text{ (for single member–single load case)} \\ &= C_p + u_n b \exp(-C_p/c) \end{aligned}$$

The derivative of the total cost is

$$\begin{aligned} dC_t/dC_p &= 1-(U_n b/c)\exp(-C_p/c) \\ &= 1-p_n U_n/c \end{aligned} \qquad [3.5]$$

Equating this to zero gives the useful result that the total cost is minimum when the expected cost of failure, $p_n U_n$ (i.e. risk) equals the attenuation cost, c, i.e. $p_{no} U_n = c$, where the subscript o refers to the optimum.

For the further development of the theory applicable to multiple failure mechanisms etc., the relevant references[3.11] should be consulted. Theoretically, the same principles could be applied in the formulation of contractual clauses covering construction tolerances on dimensions, strength and other material properties by penalizing the contractor for defective work on the basis of a loss function as an alternative to necessarily requiring replacement or strengthening of the defective portions.[3.12]

3.2.2 The Uniqueness of Bridges

The reliability analysis applied to electronic systems and small mechanical systems is a subdiscipline of mathematical statistics in that the multiple replication testing of component parts is possible. In bridges, the problem is different. Although it may be possible to proof-load component members or parts of bridge structures, the characteristics of the whole system can only very approximately be established by model testing. In the design of large structures, system reliability over and above member reliability is therefore very important as it is an elementary principle that larger systems are less reliable than smaller systems. In the context of bridges, larger systems would imply more and longer members without any substantial increase in redundancy. The development of structural reliability analysis for the design of bridge structures is thus essential if significant progress is to be made in the design of large bridges.

3.2.3 The Concept of Risk ($\Sigma p_m S_m + \Sigma p_n U_n$)

For the purposes of design, the optimal level of risk could ideally be established by maximizing the relevant utility function or, in a less comprehensive form, minimizing the loss function, C_t. As mentioned above, there are, however, practical problems that have as yet not been resolved. The idealized procedures of classical reliability theory applied to structures are not amenable to accurate solutions, largely because of the lack of statistical knowledge about the extremes (tails) of the probability density functions of the applied forces and the material properties and physical processes that determine the behaviour of structures. There are, however, various considerations that give positive guidance to the order or risk that is reasonable in terms of present socio-economic standards.

In the first place, it is an accepted principle of code theory[3.11] that new codes should be calibrated on the basis of existing standards proven by past experience so as to obtain effectively the same reliability and to adjust values only as additional knowledge and information are gained or as the socio-economic demands change.

Secondly, it has been demonstrated very convincingly by Ligtenberg[3.13] that an upper bound to practical reliability is in fact not determined by what are usually considered to be random quantities (i.e. dimensions, material properties, loads, etc.), but by other risks outside this 'normal' range of fluctuations, such as those due to gross structural errors, fires, explosions, collisions, severe corrosion and various types of abnormal or catastrophic excitation. His analysis of damage or failure due to these 'abnormal' events in the Netherlands indicates probabilities of occurrence of the order of 10^{-2} to 10^{-3} within the useful life of a building structure, as against the low figures of 10^{-4} to 10^{-6} usually assumed for structural collapse due to the random events referred to above. In the light of these studies, it would appear that the latter values are rather conservative.

However, it is clear that the profession and the general public will not readily accept a code which has as its basis a procedure whereby the risk of collapse of permanent structures such as buildings and bridges is knowingly increased relative to present standards which require that such risk shall be virtually non-existent. It can be shown that the cost of most structures is not very sensitive to variations of the probability of failure on the basis of classical reliability theory, and that the cost paid for being slightly conservative is not very great. This statement, however, must not be misinterpreted, for injudicious over-design can be very wasteful without necessarily improving safety.

Our immediate conclusion is therefore that, although reliability is of prime concern, the ultimate refinement of the design of a specific structure is not principally related to reliability, but rather to other factors such as its configuration and the materials and methods of construction, all of which have a more significant influence on the cost. In many structures, only certain members are critical in the sense that failure of a member could cause total collapse of the whole or a large portion of a structure. Failure of the balance of the members may at worst cause only local damage. It therefore stands to reason that those critical members, or combinations of members, should have lower probabilities of failure than the less critical in order to maximize the utility function (see also Section 3.2.6).

Similar arguments apply to larger members which theoretically have an increased probability of failure, and to large structures such as tall buildings and bridges consisting of a large number of members with many possible modes of failure. Redundancy, on the other hand, has the effect of reducing such risks. The importance of system reliability as against member reliability is thus clear and it may be necessary in specific cases, where a risk exceeds an acceptable limit, to modify the design so as to comply with the fail-safe principle.

Where, however, certain critical members, for instance the piers of a bridge, have a very low-level probability of over-loading due to traffic loads but may fail with catastrophic results due to the impact of a heavy vehicle or a ship in the case of bridges spanning navigable rivers, estuaries or shipping lanes, even a probability of failure based on design for normal loads as low as 10^{-6} may be meaningless unless special precautions in the form of protective structures or islands are taken to protect the piers against collision (see Fig. 5.18). Many failures of piers have also occurred because of abnormal flooding of rivers. Piers should therefore be designed more conservatively because of the increased implications of failure and it is clear that, in order to achieve more consistent levels of safety, cognizance must be taken of abnormal events (see also Chapter 2, Section 2.4.5, and Section 3.2.5). Whereas it is reasonable to reduce safety levels for elements that are less critical, it is essential to establish system reliability on a proper assessment of risk.

These principles are equally important in the design of temporary works for large bridges where the assessment of risk is very important in optimizing constructional costs. Whereas the above-mentioned optimization in terms of risk is difficult to apply because of the complexity of most practical cases, it is nevertheless possible to minimize risk at a specific cost level. This can be achieved by judicious design employing practical and economical methods of reducing the probability of failure of the structure when subjected to various actions.

As an example, this can be achieved with respect to flood actions by using the following strategies:

1. Select a structural configuration and deck level that minimize the flood action. A more robust structure with larger spans is indicated. Alternatively, where circumstances and economic factors indicate otherwise, accept the possibility of overtopping and design the structure accordingly.
2. Incorporate redundancies in the structural systems that resist the flood actions (see Section 3.2.6).
3. Design approach fills to be resistant up to a specific level of flood but allowing for overtopping and subsequent failure by wash-away at a critical level so as to reduce the forces on the bridge structure.

3.2.4 The Risk of Gross Errors

Gross errors, as distinct from random and systematic errors, will, however, remain the most serious threat to the safety of structures. The underlying causes can be roughly classified as follows:

1. A conceptual misunderstanding of one or more aspects of structural function on the part of the designer or analyst.
2. The use of incorrect assumptions as the basis for the design.
3. Gross computational errors.
4. A breakdown in communication (brief, specifications, drawings, instructions).
5. Undetected flaws in materials and serious omissions or errors in the execution of the work (workmanship).

All these errors, except those in Category 1, can be eliminated by systematic control using checking procedures. Category 1 errors, however, may require a much higher level of activity in that their elimination presupposes the recognition and complete understanding of all the critical problems and an ability to analyse and solve them in terms of the specific requirements. Major structures such as bridges should consequently only be entrusted to experienced engineers of proven ability and the designs should be subjected to design reviews and analysis by independent teams of experts.

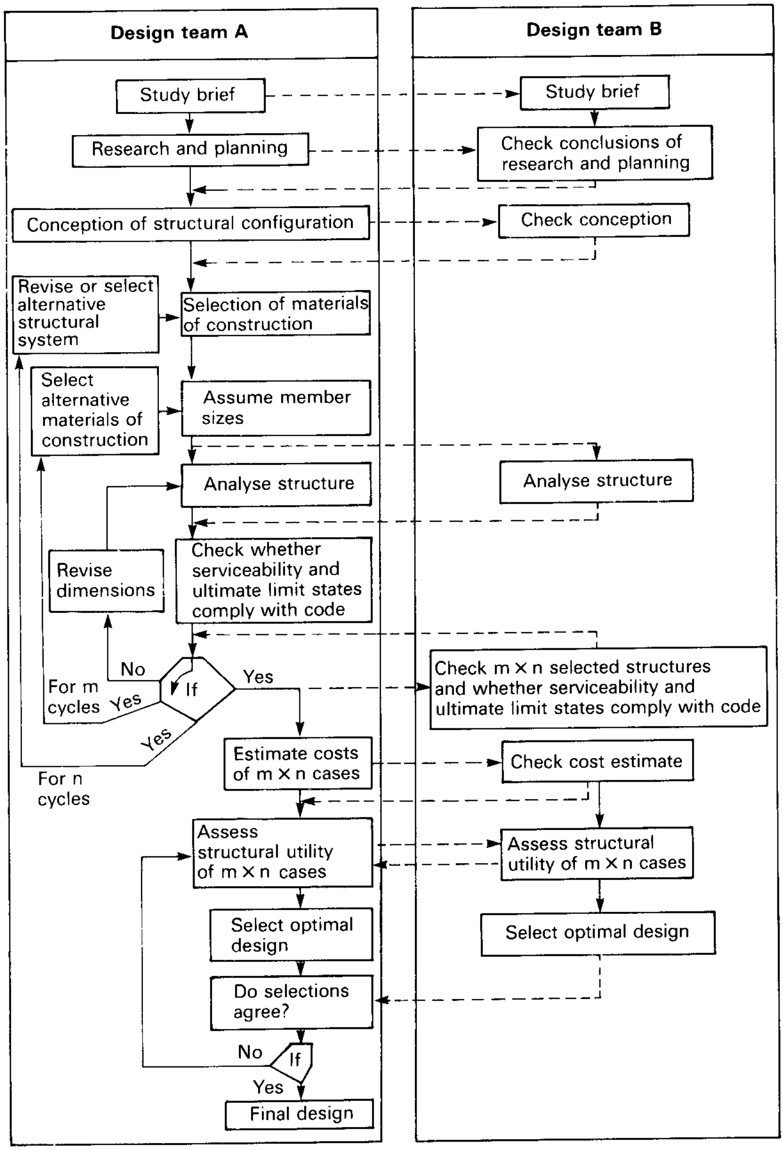


Figure 3.3 Simplified flow chart for limit state structural design with independent checking

The other categories can be eliminated by developing systematic checking and feedback procedures which are incorporated into all stages from research and planning through conception and design to the detailing and construction stages. The normal design processes can be represented by the simplified flow chart shown in Fig. 3.3.

The addition of independent checking procedures, shown in broken lines, if executed in systematic routine manner, would greatly reduce these risks. Unfortunately, these procedures can be costly and in practice some of the detailed checking procedures are reduced to design reviews of a more cursory nature by experienced personnel. This would at least reduce the risk of major conceptual errors.

If the cost of design were to be considered as another term of the utility function, it would be clear that such cost should not necessarily be proportional to the cost of the structure. In fact, increased expenditure on design reduces structural cost and risk. In the case of major structures, the latter factor may be a prime consideration.

3.2.5 Integrated Reliability Analysis of Bridges

Whereas in conventional design the various aspects of analysis are separated by specifying loads and other influences without regard to the behavioural characteristics of the structure and vice versa, reliability analysis recognizes the interrelationships that demand that an integrated analysis be carried out.

This applies particularly to dynamic effects and a sophisticated analysis which integrates the action of the applied forces with the dynamic response becomes essential if meaningful predictions of the probability of failure are to be made. Analysis at this level involves rather complex problems in the theory of stochastic processes and probabilistic structural dynamics. Considerable progress in this field has been achieved by Davenport and others, who have developed the spectral analysis method for the effects of wind forces on building structures.[3.14] The ultimate objective has been to abandon the safety-factor approach of conventional design methods based on low-occurrence intervals, which does not guarantee a consistent level of reliability, and rather to base the analysis on the response characteristics associated with a much longer recurrence interval. This load level would then be associated with a safety factor which is much closer to unity.[3.15] This equally applies to the effects of flooding and earthquake actions as discussed in Chapter 2, Sections 2.4.5 and 2.4.6 and the noted references.

In theory, it is necessary to develop probabilistic models of the various possible combinations of load actions and other influences on the structure and to establish all possible modes of failure. In bridges, these modes of failure can be very complex, consisting of brittle, ductile or mixed types of modes which may vary from extensions of a 'weakest link' model to multiload combinations, to models that may suffer collapse due to progressive conditional failures[3.16] (see also Section 3.2.6).

It is, however, usually possible to identify the more significant modes and to use a simplified model of the structure. Nevertheless, even such simplified models are far more complex than the fundamental reliability case where a load system defined by a single parameter acts on a structure with only one potential failure mode. Although the more refined models required to represent bridges can be reduced to what are conceptually fairly simple systems, their analysis remains highly complex. The main object of research in this field at present is to establish typical relationships between system and member reliability for the different levels of damage ranging from cracking and deflections to ultimate collapse, and to choose safety factors consistent with knowledge of determinate structural phenomena as well as information on random properties of loads and resistances. In the meantime, a significant advance has been made by the so-called 'engineering' approach, which is based on probabilistic concepts but which aims to maintain the simplicity of existing design codes.

3.2.6 Combinational Reliability and Redundancy

In practice some bridge structures may be complex, and global multimember analyses of the reliability of such structures would be necessary for accurate results. It is hardly practical to do the required analyses as the procedures given in codes of practice for determining limit states are semi-probabilistic based on a single-element approach and, except for very simple structures, the required procedures are very complex. Nevertheless it is useful to consider the combinational effects of multiple-member and redundant systems on the reliability as well as the statistical dependence (correlation) between mode failure events. In practice the extreme effects of full correlation and independence would be interpolated by judgement. Variance in the resistance of concrete members at the ultimate limit state will furthermore be affected by member size, depending on the size of batches of concrete placed. Large members cast in small batches will tend to have larger variance at a section than smaller members cast from larger batches, but the strength at any section of the former will tend to approximate more closely with the mean strength. The replacement of sub-standard concrete identified by testing would reduce the risk very substantially due to its large influence on the convolution integral of the probability density function of action and the cumulative distribution function of resistance (see Figs 3.1 and 3.4). The systematic application of well-managed maintenance procedures will likewise reduce the risk by the pre-emptive repair of deteriorating members (see Chapter 6).

By assessing the combinational reliability[3.17] of a structure or portions of the structure, even in an approximate manner, it is possible to reduce the risk of failure substantially by optimizing the number and arrangement of elements. If we consider a bridge pier founded on piles which is a combination of a series ('weakest link') system and parallel systems (piles), we can express the bounds of probability of failure (p_f), when

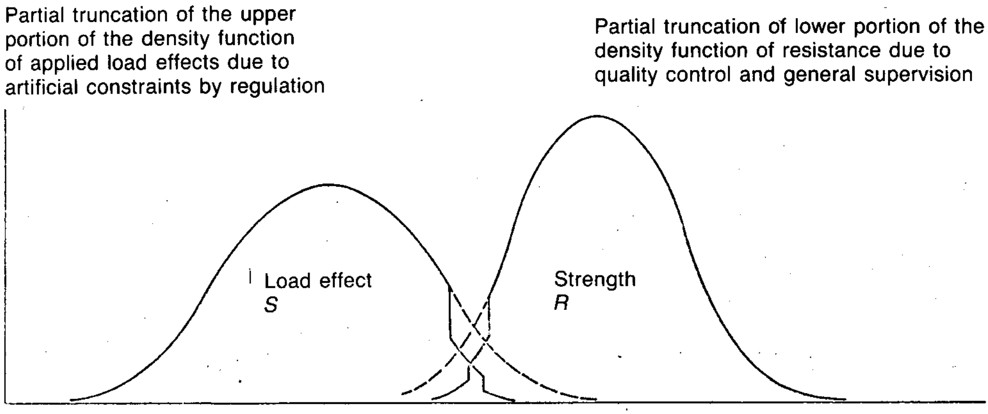


Figure 3.4 The effect of control on the probability of failure

it is subjected to a deterministically calculated flood action which is equal to the design load multiplied by the partial load factor, approximately as follows.

Assuming no statistical dependence between components, the upper bounds are

$$p_f \leq 1 - \{1 - p_{\text{pier}}\}\{1 - p_{\text{cap}}\}\{1 - p_{\text{pile group}}\} \qquad [3.6]$$

where

$$\{1 - p_{\text{pile group}}\} = \left\{ \sum_{i=r}^{n} \binom{n}{i} (1 - p_{\text{pile}})^i (p_{\text{pile}})^{n-i} \right\} \{1 - p_{\text{sg}}\} \qquad [3.7]$$

where

$$(1 - p_{\text{pile}}) = (1 - p_{\text{cp}})(1 - p_{\text{sp}}) \qquad [3.8]$$

and

$$p_{\text{pile}} = p_{\text{cp}} + p_{\text{sp}} - p_{\text{cp}} p_{\text{sp}} \qquad [3.9]$$

where

p_{cp} = probability of failure of a pile depending on its load

p_{sp} = probability of local failure of the sub-soil supporting individual piles

p_{sg} = probability of failure of the sub-soil supporting the pile group as a whole

n = number of piles in the group

r = minimum number of piles required to ensure statical equilibrium; and

$$\binom{n}{i} = \frac{n!}{i!(n-i)!} \quad \text{where } ! = \text{factorial}$$

The lower bound of p_f assuming 'plastic' failure modes is greater than or equal to Eq. 3.6 but with

$$\{1-p_{\text{pile group}}\} = \left\{\left(1-\prod_{i=r}^{n} p^{i}_{\text{pile}}\right)(1-p_{\text{sg}})\right\} \qquad [3.10]$$

where

$$\Pi = \text{product of}$$

For structures designed to specific codes the order of p_{pier}, p_{cap}, p_{cp}, p_{sp}, and p_{sg} is usually between 10^{-3} and 10^{-6} and the values of p_{cp} and p_{sp} would furthermore depend on the load on individual piles; and the correct value, depending on the failure modes, would lie between the upper and lower bounds. These bounds would be affected if there is statistical correlation between components, which can be expressed by reformulation using a covariance matrix. The bounds indicated are in practical terms trivial and the reader is referred to a paper by Ditlevsen and Bjerager[3.18] for a more comprehensive treatment. Monte Carlo simulation is the only practicable way of obtaining meaningful estimates of the probability of failure, especially if the probabilities of the load effects are also taken into account.

However, the matter that is of interest here is to deduce the effect of additional piles in excess of r in number. By substituting any reasonable values for the various probabilities of failure of elements in Eqs 3.6 and 3.10 it can be shown that whereas p_f increases as r approaches n, there is a marked decrease for increased n. The exponential terms, e.g. $(1-p_{\text{pile}})^i$ and $(p_{\text{pile}})^{n-i}$ have to be determined for the relevant p_{pile} values of each pile, depending on its load. These values of p_{pile} are much reduced at reduced load levels which increases the reliability additionally to that due to redundancy.

The influence of redundancy on the reliability of structures has been discussed by various authors.[3.19,3.20] Fred Moses has demonstrated this in a very simple manner as follows. Consider an encastred beam subjected to midspan point load. Failure occurs if the beam reaches its bending moment capacity at the support points and at the load point. The margin of safety, using plastic analysis, can be written as

$$\text{M.S.} = M_{\text{S1}}+M_{\text{m}}+M_{\text{S2}}-WL/2 \qquad [3.11]$$

where M represents the various moment capacities. If the moment capacities are statistically independent, then

$$m_{\text{M.S.}} = m_{M_{\text{S1}}}+2m_{M_{\text{m}}}+m_{M_{\text{S2}}}-m_{\text{W}}L/2$$

and

$$\sigma^2_{\text{M.S.}} = \sigma^2_{M_{\text{S1}}}+4\sigma^2_{M_{\text{m}}}+\sigma^2_{M_{\text{S2}}}+\frac{L^2}{4}\sigma^2_{\text{W}}$$

where m indicates mean values and σ is the standard deviation.

If we look just at the resistance portion R and assume that the moment capacities have equal means m_M and standard deviations σ_m:

$$\sigma^2_R = 6\sigma^2_M$$

The coefficient of variation of the resistance is

$$V_R = \frac{\sqrt{6\sigma^2_M}}{4m_M} = 0.61V_M \quad [3.12]$$

Thus the coefficient of variation of total resistance is reduced below that of an element. This is because low strength of one element can be counterbalanced by high strength of another element and the two add together to the resistance, provided the system is ductile. However, if the moment capacities are not statistically independent but are completely correlated,

$$V_R = \frac{\sqrt{16\sigma^2_M}}{4m_M} = V_M \quad [3.13]$$

and there is no change in the coefficients of the variation of the strength of the system compared with the elements. However, this is rare and redundant structures usually have an additional reserve of resistance at the ultimate limit state if designed for the envelope of forces or moments due to variable actions. The configuration of the structure is important, e.g. the benefit of a continuous deck acting in conjunction with the piers and abutments to resist flood forces acting on the substructure is obvious.

3.3 Codified Design

The object of codes of practice should be to maximize utility from the viewpoint of society but, for the reasons given above, the present state of knowledge is such that interim codes will have to be expressed in less advanced terms in such a manner as to facilitate their evolution towards a more ideal form.

The CEB model code[3.21] and the practical codes such as the British Standards Institution's CP 110: 1972, *The Structural Use of Concrete* (amended to November 1980), and BS 5400: Parts 1–10, (1978–84)[3.22] are Level 1 semi-probabilistic codes based on the limit state design method. These codes represent an advance in that the uncertainty of design data is recognized and expressed in statistical terms and the use of different partial factors to check the structure for the serviceability and ultimate limit states under various conditions of loading, and other influences, reduces the inconsistencies that are apparent in the working stress methods of previous codes. The theoretical basis of the above codes, and the methods of calibrating the partial factors against existing codes to ensure that the new codes do not allow a lower reliability than the least acceptable in current practice, are explained to considerable depth in the CIRIA Report 63 of July 1977[3.9] on the *Rationalization of Safety and Serviceability Factors in Structural Codes*. The calibration process consists of rationally selecting the target value of probability to be used in deriving the safety factors for the new code. The probability is 'notional' since it represents only the influence on safety of the variability of loading and resistance and because of certain approximations made in the calculations of the probability. The total probability is also influenced by the risks of gross errors which cannot be adequately quantified for a specific case and which are only partially compensated for by conservative assumptions. It is furthermore reduced by the fact that structural redundancy is ignored in the calibration process. Such procedures do, however, ensure a continuity of confidence and benefit from past experience. There are nevertheless shortcomings. The characteristic or nominal values for the material strengths and design forces, which are arbitrarily defined in terms of normal distributions, do not necessarily ensure a consistent level of reliability, especially in the case of certain members subjected to stress reversals. The partial factors were introduced largely to differentiate between influences that exhibit greatly varying margins of safety and which could not be covered adequately as well as consistently by a single global factor. This is, however, a great improvement in that satisfactory levels of safety are ensured.

A study of recent publications[3.19,3.20] indicates that there has been considerable progress in the conceptual understanding and theoretical treatment of the various problems touched on above. However, the practical application of the procedures are difficult and some fundamental problems remain intractable. Fred Moses[3.19] has outlined a practical method of identifying collapse mode expressions for large structural systems which incorporate both ductile and brittle components. The method utilizes existing structural analysis programs with incremental loadings and reanalyses following successive redundant component failures. These procedures have not been fully developed in some fields such as in geotechnical design where there are many unresolved problems. The design of bridges against the actions of floods is another case that needs urgent attention (see Chapter 2, Section 2.4.5).

The Joint Committee on Structural Safety[3.23] supported by CEB, CIB, ECCS, FIP, IABSE, IASS and RILEM has recently produced a working document on proposals for a code for the direct use of reliability methods in structural design, as a first step towards implementation. It is the general opinion of the JCSS Working Party that reliability methods have advanced to such an extent that they may not only be used for deriving safety provisions in codes. However, a design which utilizes the full statistical information available and the advantages of a direct probabilistic modelling is possible, if only relevant for special situations. It is well understood that this type of code will never replace present (deterministic or reliability-based) codes. However, it may serve as a fundamental code which is supplemented by codes giving rules for common design.

3.4 Modern Codes of Practice for Bridges

Codes of practice prepared by various authorities in different countries and intended for general use in the design and construction of concrete structures, while mainly orientated towards building structures, are discussed and compared by Rygol in Chapter 40 of Ref. 3.24.

Some authorities responsible for the construction of bridges, or institutes promoting the use of concrete, have accordingly produced specialized codes such as the *Standard Specifications for Highway Bridges* (12th edition, 1977) adopted by the American Association of State Highway and Transportation Officials and the *ACI Manual of Concrete Practice: Bridge Analysis and Design* (ACI 343R-77) in the USA. In the United Kingdom, the BSI has published *BS 5400*, Steel, concrete and composite bridges: *Parts 1–10 (1978–1984)*.[3.22] In the case of the aforementioned AASHTO and ACI codes, reinforced concrete members are designed either with reference to service loads and allowable stresses or, alternatively, with reference to load factors and nominal strengths multiplied by strength (or capacity) reduction factors. The capacity reduction factors depend on the statistical variance of the type of resistance being considered.

The codes that are currently applicable in some countries are still based on the working-stress or factor-of-safety method, with a deterministic philosophy. However, most authorities have reviewed the situation or are in the process of doing so in the light of proposals such as that of the CEB model code. The UK has been the forerunner in this respect, by producing BS 5400. It consists of ten parts:

1. General statement.
2. Specification for loads.
3. Code of practice for design of steel bridges.
4. Code of practice for design of concrete bridges.
5. Code of practice for design of composite bridges.
6. Specification for materials and workmanship, steel.
7. Specification for materials and workmanship, concrete, reinforcement and prestressing tendons.
8. Recommendations for materials and workmanship, concrete, reinforcement and prestressing tendons.
9. Code of practice for hearings.
10. Code of practice for fatigue.

As a code it is based on the same philosophy and in detail is similar to CP 110, which is a great advance on previous codes. These codes were initially not readily accepted by the practising fraternity in general because of the increased complexity, and have not immediately resulted in more economic structures because of the calibration procedures used for the partial factors, as explained in Section 3.2. There are differences of opinion of the consistency of certain design procedures, such as those for determining the shear and torsional resistance, which are largely empirical and give very different results from the more recent procedures proposed by Thürlimann.[3.25,3.26]

The CEB model code allows the use of both methods as alternatives, apparently reflecting a difference of opinion amongst the responsible committee members. The so-called 'accurate' approach is based on a simplified theoretical model of beams which are considered to consist of discrete but contiguous compressive members acting in the approximate direction of the principal compressive stresses to form a highly redundant shear or torsional truss system in conjunction with tensile reinforcement or prestressing. Compatibility equations for the simplified systems are satisfied.

In the case of Europe, the formation of the European Economic Community (EEC) is probably going to result in a more uniform approach in the future in the form of Eurocodes and supporting European standards published by the European Committee for Standardization (CEN) and based on international standards wherever possible in the replacement of existing national codes and standards.

It is highly probable that the more general acceptance of a common philosophy and improved communication internationally will result in codes of practice becoming more similar in the future.

References

3.1 Lovelace A 1969 Keynote address *Proc. Int. Conf. Struct. Safety and Reliability* Smithsonian Institute, Washington DC, April 1969, pp 3–4

3.2 Pugsley A G 1966, *The Safety of Structures* Arnold, London, 156 pp

3.3 Freudenthal A M Mar 1961 Safety, reliability and structural design *J. Struct. Div. Am. Soc. Civ. Eng.* **87** (ST3): 1–16

3.4 Ang A H S 1969 Probability considerations in design and formulation of safety factors *Proc. Symp. Concepts of Safety of Structures and Methods of Design* International Association for Bridge and Structural Engineering, London, pp 13–23

3.5 Cornell C A 1969 Bayesian statistical decision theory and reliability-based design *Proc. Int. Conf. Struct. Safety and Reliability* Smithsonian Institute, Washington, DC, April 1969, pp 47–68

3.6 Cornell C A 1969 Structural safety specifications based on second-moment reliability analysis *Proc. Symp. Concepts of Safety of Structures and*

Methods of Design International Association for Bridge and Structural Engineering, London, 1969, pp 235–45

3.7 Milford R V 1988 Target safety and SABS 0160 load factors. *The Civil Engineer in South Africa, Trans. SAICE* **30** (10): 475–81

3.8 Esteva L 1972 Summary report, Technical Committee No. 10: Structural safety and probabilistic methods *Proc. Int. Conf. Planning and Design Tall Bldgs* vol 1b, Lehigh University, Bethlehem, Pennsylvania, Aug 1972, pp 1043–66

3.9 Construction Industry Research and Information Association Jul 1977 *Rationalization of Safety and Serviceability Factors in Structural Codes* CIRIA report 63, London, 226 pp

3.10 Turkstra C J 1970 *Applications of Bayesian Decision Theory: Structural Reliability and Codified Design* SM study no 3, University of Waterloo, Solid Mechanics Division, Ontario, Chapter 4, pp 49–71

3.11 Lind N C, Davenport A G 1972 Towards practical application of structural reliability theory: probabilistic design of reinforced concrete buildings, American Concrete Institute, SP-31, Detroit, Michigan, pp 63–110

3.12 Rosenblueth E 1972 Technical Committee no 10, Report no 2, Code specifications of safety and serviceability *Proc. Int. Conf. Planning and Design Tall Bldgs* vol 1b, Lehigh University, Bethlehem, Pennsylvania, Aug 1972, pp 931–59

3.13 Ligtenberg F K 1969 Structural safety and catastrophic events, *Final Report Symp. Concepts for Safety of Structures and Methods of Design* International Association for Bridge and Structural Engineering, London, pp 25–33

3.14 Davenport A G 1966 The treatment of wind loading on tall buildings *Proc. Symp. Tall Bldgs* Dept Civil Engineering, University of Southampton, April 1966, pp 3–44

3.15 Davenport A G 1969 Structural safety and reliability under wind action *Proc. Int. Conf. Struct. Safety and Reliability* Smithsonian Institute, Washington, DC, April 1969, pp 131–145

3.16 Moses F and Tichy M 1972 Safety analysis for tall buildings *Proc. Int. Conf. Planning and Design Tall Bldgs* Lehigh University, Bethlehem, Pennsylvania, Aug 1972, Technical Committee no 10, State-of-the-art report no 5, pp 993–1005

3.17 Harr M E 1977 *Mechanics of Particulate Media — Combinational Reliability* McGraw-Hill International, New York, p 408

3.18 Ditlevson O and Bjerager P 1986 Methods of structural systems reliability. In *Structural Safety* vol 3, nos 3 and 4 (special issue), Elsevier Science, Amsterdam, pp 195–229

3.19 Moses F 1982 System reliability developments in structural engineering. In *Structural Safety* vol 1, Elsevier Science, Amsterdam, pp 3–13

3.20 Hohenbichler M, Rackwitz R 1983 First order concepts in system reliability. *Structural Safety* vol 1, no 3, Elsevier Science, Amsterdam, 177–88

3.21 Comité Euro-International du Béton (CEB) 1978 *International System of Unified Standard Codes of Practice for Structures* vol I: *Common Unified Rules for Different Types of Construction and Material* 123 pp, and vol

II: *CEB–FIP Model Code for Concrete Structures* Bulletin d'information, N.124/125-E, 348 pp 1970

3.22 British Standards Institution *BS 5400: Parts 1–10: 1978–1984* Steel, concrete and composite bridges

3.23 Ditlevsen O, Madsen H O Nov 1989 *Proposal for a Code for the Direct Use of Reliability Methods in Structural Design* Working document, IABSE, Zürich

3.24 Kong F K, Evans R H, Cohen E, Roll F (eds) 1983 *Handbook of Structural Concrete* Pitman, London

3.25 Lampert P, Thürlimann B 1971 Ultimate strength and design of reinforced concrete beams in torsion and bending *IABSE Publications* **31–I:** 107–31

3.26 Grob J, Thürlimann B 1976 Ultimate strength and design of reinforced concrete beams under bending and shear *IABSE publications* **36–II:** 105–20

4 Methods of Analysis of Bridge Structures

4.1 General

A brief account is given in Chapter 1, Section 1.2, of the historical development of the capacity of engineers to analyse bridge structures, from the early crude efforts based largely on intuition, to the more systematic empirical procedures which were developed rather spasmodically during specific periods of civilization by the accumulation of rules based on experience and tests. The early engineers were practical men who failed to benefit fully from such primitive theories as were developed by their contemporaries amongst the philosophers. However, the mathematicians and physicists of the 17th century and onwards played a major role in producing theoretical formulations that eventually made it possible, by the end of the 19th century, for bridges to be designed and analysed almost entirely on the basis of theoretical methods using parameters for material properties and applied forces that had been predetermined by experiment. Initially, many of these theories had been grossly over-simplified abstractions which at best gave only very approximate solutions. The range of practical configurations was accordingly limited and it was common practice to make structures statically determinate by the insertion of hinged joints or other discontinuities in order to simplify the analysis.

Following on the initial developments referred to above, the theory of elasticity, as applied to structures, developed rapidly and solutions were found for many of the relatively complex problems presented by structures with redundant members. The most significant development in the linear structural analysis of statically indeterminate structures has been the influence coefficient method based on the work of Clerk Maxwell, Alberto Castigliano (1847–1884) and especially H. Müller-Breslau in Germany (1886) and Camillo Guidi[4.1] in Italy. The extreme complexity of many types of structure has, however, provided serious barriers to progress because of the intractability of the mathematical equations of analysis, and engineers were forced to rely on greatly over-simplified theories in order to obtain workable solutions. It is a historical fact that engineers have seldom balked at any problem that barred progress for lack of precise theories of analysis, but they have unfortunately also paid a heavy price in failures. Venturing into bigger and unknown terrain by extrapolations is extremely risky without a complete understanding of the relevant problems. Excluding cases of gross negligence, almost all recent major collapses fell into this class of error. Many of these were due to effects which in terms of simplified theories were considered to be of secondary importance but actually resulted in serious overstressing or instability.

The advent of numerical, and especially iterative, methods, such as the moment distribution methods of Hardy Cross and the relaxation methods of Southwell, introduced a new era to analysis. Whereas the period preceding these innovations was characterized by a great variety of special methods, each of which provided the best solution for a particular problem, it was now possible to adopt a more general method. Although the advent of the electronic computer in the 1950s gave this approach tremendous momentum, the more significant development resulting therefrom was the return to a generalized mathematical formulation of equations, which had now become feasible for extremely complex problems. The matrix algebra which had been developed almost a century earlier by Arthur Cayley, as an exercise in pure mathematics, presented an ideal means of formulating problems for processing by computer and, together with the very compact notation of modern mathematics and the invariance principle as used in tensor analysis, has been a major advance in obtaining general methods that are applicable to a much greater range of problems. Whereas initially most methods of analysis were formulated in terms of flexibility matrices, the advent of large-capacity computers has made the more powerful stiffness approach feasible.

The natural result of these developments was the emergence of integrated engineering computer systems[4.2] which provide a very comprehensive facility for the solution of interrelated problems. In spite of the inherent advantages of these programs, a considerable number of special programs are being written for repetitive problems because of the increased economy or because of the specific nature of the problem. Whereas the aforementioned systems, which operate on large-capacity computers, are used to solve complex problems, the bulk of engineering design problems encountered in bridges can be effectively processed on the minicomputers or self-contained programmable calculators (see Chapter 22 of Ref. 4.3).

Whereas the mathematical models which served as abstract representations of structural behaviour in the past were in many ways over-simplified because of the limited means of analysis available, closer approximations to the prototype are now feasible with the application of methods such as the finite element and finite strip methods, of which there are several versions available and which are developing very rapidly in scope and refinement. These refinements, in conjunction with large-capacity computers, enable the non-linear analysis of the structural behaviour of complex three-dimensional members and structural systems.[4.4] Further refinements include the more accurate simulation of the non-linear behaviour of concrete as a non-homogeneous material, such as bond and anchorage slip and the formation of cracks, by discrete or smeared models using suitable linkage elements as well as the effects of stirrups, binders, dowel shear and aggregate interlock.[4.5] The limit of accuracy of this technique is at present only dependent on available computer capacity and the economics of analysis. Likewise the application of interactive graphics and computer-aided design (CAD) is eliminating many man-hours of the analytical stages of design and detailing as at present constituted. Knowledge-based expert systems will no doubt play an ever-increasing role in the improvement of design optimization.

It is, however, generally accepted practice to use linear elastic methods in the analysis of bridge structures. These results can readily be applied to the serviceability condition of limit state analysis, but require appropriate factoring for the load effects of the various load combinations for the ultimate limit state. For critical cases it may be necessary to do non-linear analyses, e.g. in cases where buckling or large deflections may generate additional stresses or cause a member to approach instability. Where the assumption of partial or total plasticity at cross-sections where maximum stress conditions occur implies a more general non-linear behaviour of the structure, it may require further investigation.

4.2 Mathematical Simulation of Material Properties

The basic assumptions which determine the mathematical properties (constitutive equations) of structural concrete and steel for use in practical design are usually defined in codes of practice issued by the responsible authority. These properties relate *inter alia* to the stress–strain relations and limits, the effects of cracking of concrete, and the modes of failure and the yield and ultimate strengths of steel and concrete respectively. The assumptions and properties recommended by the CEB–FIP are described in the *Model Code for Concrete Structures: Structural Design and Detailing*.[4.6] For further information relevant to concrete bridges, see Refs 4.6–4.11 and Chapter 12 in Ref. 4.3.

For linear elastic analysis of structures subjected to static or quasi-static loading of short duration, it is accepted practice to use the elastic concrete properties of the monolithic concrete to determine the member stiffnesses for both the serviceability and ultimate limit states. The effective Young's modulus of concrete used for analysis is taken to be the secant modulus, which is approximately

$$E_{sec} = 0.9\,E_c \qquad [4.1]$$

where E_c is the initial tangent modulus of concrete.

The torsional stiffness, k_t, of bridge deck members can play an important role in the analysis of bridge structures and must be given more consideration than is normally required for beam members in buildings. Due to the presence of microcracks, the torsional stiffness is substantially reduced relative to the elastic stiffness of the section, unlike the flexural stiffness which may only reduce by a few per cent when the tensile zone of the concrete starts to develop cracks. This reduction in stiffness can manifest itself already at the serviceability level of loading and is fully developed at an ultimate limit state level of loading. Using the formulation for torsional stiffness

$$k_t = GJ/L \qquad [4.2]$$

where G is the shear modulus, J the torsional rigidity of the section in the uncracked state, and L the length of the member.

The CEB–FIP model code recommends the following values for G in the absence of more accurate methods:

$$G_{\mathrm{I}} = 0.30\,E_{sec}/(1+1.0\phi) \qquad [4.3]$$
$$G_{\mathrm{II}_m} = 0.10\,E_{sec}/(1+0.3\phi) \qquad [4.4]$$
$$G_{\mathrm{II}_t} = 0.05\,E_{sec}/(1+0.3\phi) \qquad [4.5]$$

where subscript I refers to the uncracked state of the section, II_m to the state when flexural cracks have formed, and II_t to the state when torsional and shear cracks have developed; E_{sec} is the secant modulus defined in Eqn 4.1 and ϕ the coefficient of creep of concrete to be used for long-term loading.

When the level of torsional and longitudinal reinforcement is known, a more accurate expression can be used for state II:

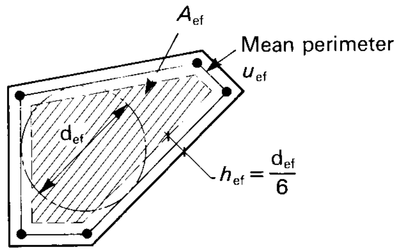


Figure 4.1 Equivalent hollow section

$$G_{II}J = \frac{E_s A_{ef}^2}{\dfrac{u_{ef}S}{2A_s} + 1.5\dfrac{E_s}{E_{sec}}\dfrac{u_{ef}}{h_{ef}}(1+\phi)} \qquad [4.6]$$

where E_s is Young's modulus of the reinforcing steel, A_s the area of a closed stirrup or a fraction of that area which balances the torsional moment, and s the stirrup spacing, and referring to Fig. 4.1 the equivalent hollow section properties are defined as:

A_{ef} = area enclosed by the polygonal perimeter length, u_{ef}, which is formed by joining the centres of the longitudinal reinforcement which are enclosed by the torsional stirrup (A_s)
h_{ef} = $d_{ef}/6$ = equivalent thin-walled thickness
d_{ef} = the diameter of the largest circle which can be inscribed within the boundaries of A_{ef}.

Torsional resistance of bridge decks in many cases are a combination of torsional shear (de St Venant twist) generated by the torsional shear stiffness of solid beams or closed hollow box beams as described above, and warping action which results from the flexural twisting of parallel longitudinal beams with axes parallel to the torsional axis and connected transversely by the deck slab and transverse beams. Warping action is most relevant for bridge decks having an open cross-section. This latter action results in the generation of a vertical bimoment[4.12,4.13] (i.e. about a horizontal transverse axis relative to the deck) in the case of two parallel beams, and in bimoment sets in the case of more than three longitudinal beams. With correct modelling (Section 4.3) the above effects are automatically accounted for.

For linear elastic analysis of structures subjected to dynamic loading, the dynamic modulus should be used. This approximates to

$$E_{dyn} = 1.25\, E_{sec} \qquad [4.7]$$

When the loading is of a long-term nature or when stress-dependent material non-linearity affects the member stiffness, then the non-linear properties of concrete are required. References 4.6, 4.8,4.9 deal with this matter in detail. The equivalent long-term modulus generally takes the form

$$E_{long} = E_t = E_c/(1+\phi_t) \qquad [4.8]$$

where ϕ_t is the creep factor of concrete.

Generally, the non-linear material properties have to be assessed for each member at each load and/or time step of the non-linear analysis in order to update the member stiffness coefficients. The relevant analytical procedures are dealt with in Sections 4.3 and 4.5. In the case where non-linear material properties are used for analysis, it is important that the unfactored stress–strain relationship be used, i.e. the characteristic or nominal values.

In some types of concrete structure, the contractor prefers to use a concrete mix which gives a 28-day strength considerably in excess of the specified characteristic strength in order to achieve the high early strengths he requires to save construction cycle time (incrementally launched decks, *in situ* segmental construction, columns cast by sliding formwork, etc.). Cognisance has to be taken of these practices in order to assess realistic material properties for analytical purposes, especially when the displacements of the structure are required. The design engineer has to be aware of the fact that displacements are proportional to the absolute values of the structural stiffness while the stress resultants (in an indeterminate structure) are dependent on the relative structural stiffness of the respective structural elements.

In finite element analysis, additional concrete properties are required.[4.14,4.67] Especially in the cases of triaxial stress problems and anisotropy, specialist literature should be consulted for the required properties and their mathematical simulation.

4.3 Mathematical Modelling of the Structural Components and Discretization of Bridge Structures

In the mathematical modelling of structural components, it is current practice to obtain discretized analytical elements that deform the same way when subjected to loading as the real structural component. This implies that the model element must have the same stiffness properties as the structural member.

When it comes to the modelling of the whole structure, the manner of discretization of the structural members into an adequate number of elements has to be decided. The following factors influence this decision:

(a) Type of structural configuration, i.e. arch structure, continuous box-girder on monolithic supports, etc., and the nature of the interaction between members.
(b) Type of loading to which the structure will be subjected, i.e. dead and live loads acting in only one plane, or earthquake loading in any direction.
(c) The nature of output required, i.e. longitudinal beam action only or beam action and transverse slab bending or stress concentrations, etc.
(d) Number of output points required. Most programs supply stress resultants only at the ends of members, so that finer subdivision may be necessary for particular load types (i.e. prestress).

(e) In finite element analysis, it must be determined how many elements are required for a sufficient solution. Unless the particular element types have been used before, it is essential to test a similar mesh configuration with known results before using them in the bridge model.
(f) More elements usually require more modes, which implies that the analytical model has more degrees of freedom. This has a direct influence on the cost of the analysis and must accordingly be optimized relative to the degree of accuracy thereof.

In order to illustrate the choice of discretization and modelling options, the structure shown in Fig. 4.2(a) has been simulated by several valid models as shown in Figs 4.2(b)–(e). The advantages and shortcomings of each of these models can easily be verified in terms of the factors listed above.

The model Fig. 4.2(b) could not correctly simulate the bridge shown in resisting torsion on the whole cross-section as it would warp whereas the three-webbed girder with an interconnecting bottom flange would mainly be subjected to distortion and St Venant shear in the closed box cross-section (see Section 4.2 and Ref. 4.16).

In the case where many different types of action are imposed on a bridge structure, as is often the case, it may seem advantageous to use several different models of the structural configuration best suited for the purpose of analysing the effects of a particular type of action. The limit state design method, however, requires an investigation of many different combinations of the respective stress resultants. Instead of this, it is often advisable rather to use an all-encompassing model and to subject it to all the different types of loading. The stress resultants can then be readily combined by the computer program as required for design purposes, provided that elastic behaviour is assumed.

In some cases of modelling it may be necessary to use hypothetical elements which ensure the correct global stiffness relationship but display meaningless local stress resultants, as in the model Fig. 4.2(b), where the total torsional stiffnesses of the assembly of three grillage members must be equated to that of the three-webbed closed box-grider. Another typical example of such a model element, encountered in grillage analysis of box-type bridge decks,[4.8,4.16] is where the transverse stiffness of the combined top and bottom slabs of the box section also has to be modelled by a single member. The required inertia for rotational stiffness and deflection (shear) stiffness, however, are quite different. Even so, the correct stiffness can be achieved by using an equivalent inertia and appropriate shear area in the modelled element. It is often necessary, in the case of such a 'coupled' element model, to resort to a small equivalent member analysis to ensure correct displacement (or stiffness) characteristics. This is shown below by way of example.

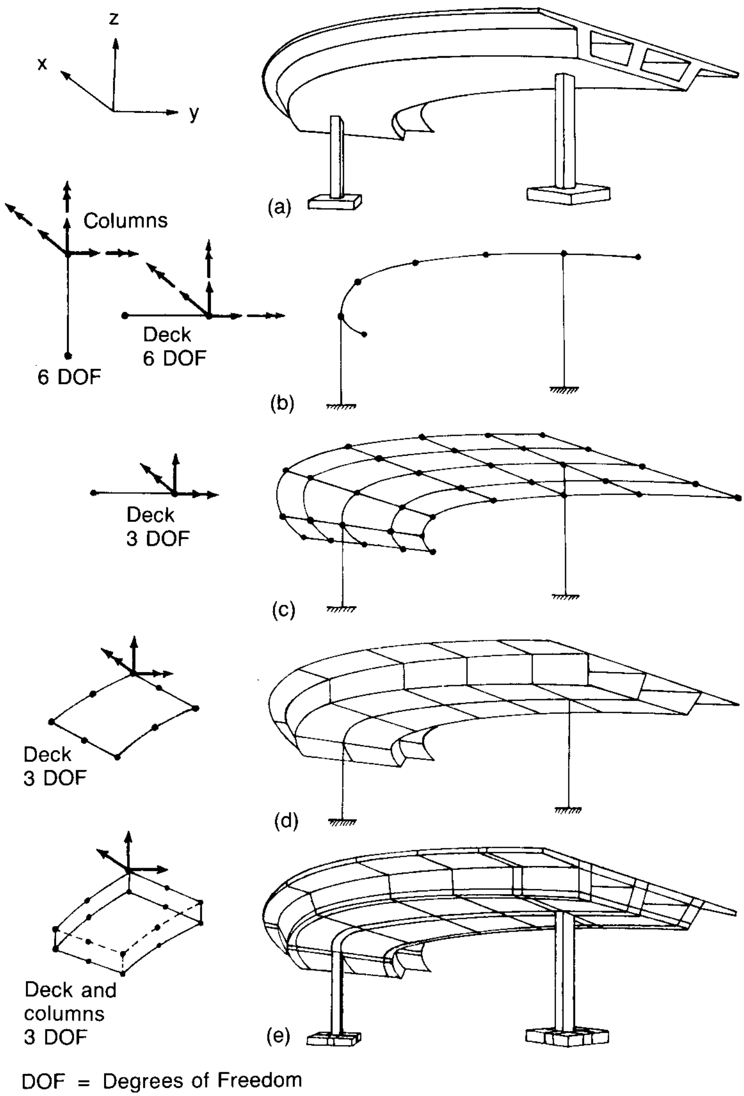


Figure 4.2 Choice of structure discretization and modelling showing degrees of freedom at the nodes

Figure 4.3(a) shows a typical single-cell box cross-section. Figures 4.3(b), (c) show the skeletal structure required to evaluate the stiffnesses for transverse 'plane-frame' deformations. The structure in Fig. 4.3(b) is *fully* fixed at both supports, except for the rotational release at support 'a' at which a moment M is applied. The resultant rotational stiffness is found from the expression

$$k_r = 4EI_r/L = M/\theta \tag{4.9}$$

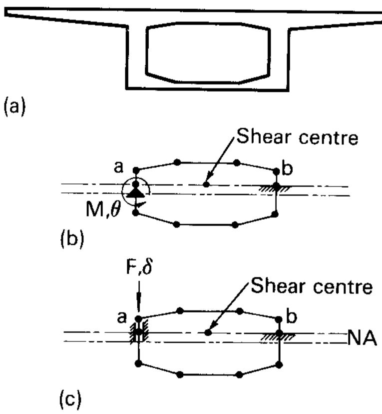


Figure 4.3
(a) typical single-box cross-section;
(b) evaluation of rotation stiffness;
(c) evaluation of deflection stiffnes

where E is Young's modulus, I_r the effective sectional inertia for an applied rotation, L the length of member between effective support points, and θ the rotation of support 'a' due to an applied moment M. Hence I_r is evaluated.

The structure in Fig. 4.3(c) is *fully* fixed at both supports except for the deflection release at support 'a' at which a force F is applied, as shown in Fig. 4.3(c). The resultant deflection stiffness is found from the expression

$$k_d = 12EI_d/L^3 = F/\delta \qquad [4.10]$$

where I_d is the effective sectional inertia for an applied deflection, and δ the deflection of support 'a' at right angles to the member axis due to a force F applied in the same direction. Hence I_d is evaluated.

Since most frame analysis programs do not allow for rotational *and* displacement inertia, but do allow for shear area, an equivalent set of inertia and shear area can be evaluated as follows.

Allowance for shear area in the stiffness matrix generally takes the form

$$k_{a_i} = 4\frac{EI}{L}\lambda_i \qquad [4.11]$$

$$k_{a_j} = 4\frac{EI}{L}\lambda_j \qquad [4.12]$$

$$k_{b_{ij}} = 2\frac{EI}{L}\lambda_{ij} \qquad [4.13]$$

where i and j refer to the supports at ends i and j of the member respectively and a and b are the diagonal and off-diagonal position indicators of the independent bending stiffness terms of the member matrix.

For prismatic members,

$$\lambda_i = \lambda_j = \frac{B+1}{4B+1}$$

$$\lambda_{ij} = \frac{1-2B}{4B+1}$$

where $B = 9I/A_{sh}L^2$ (and $B \to 0$ as $A_{sh} \to 0$, not ∞!), in which A_{sh} is the shear area.

For equivalent properties,

$$B = I_r/I_d - 1$$

and hence λ_i and λ_{ij} can be determined.

In programs where the coefficient λ cannot be input directly, an equivalent inertia and shear area can be evaluated, that is

$$I_{eq} = 4I_r - I_d$$

and

$$A_{eq}\ 9I_{eq}/BL^2$$

Specialized bridge structures such as arch bridges, strut-frames and cable-stayed bridges require more detailed treatment in modelling and discretization for both the complete structure and the various construction stages. Some of these requirements will be discussed in Sections 4.4–4.6.

Few design codes give guidance in modelling[4.6,4.7] and it is therefore advisable to refer to the modern literature for guidance.[4.11,4.15,4.17–4.24]

4.4 The Simulation of Actions on Bridge Structures

The actions to which bridges may be subjected are described and classified in Chapter 2, Section 2.4. Chapters 11, 15, 19, 20 and 23 of Ref. 4.3 should also be consulted, and Refs 4.8,4.16 and 4.25–4.31 may prove useful. For the purpose of analysis, the following re-classification is useful:

(a) *Long-term actions*. These consist of dead load, superimposed dead load, earth pressure, water pressure, long-term restraint actions due to creep and shrinkage effects, secondary hyperstatic (parasitic) prestress and prestrain effects, and the reactions and differential settlement due to foundation subsoil.

(b) *Short-term quasi-static actions*. These consist of all those short-term and variable loads which have negligible dynamic effects on a bridge structure. The following types of action normally fall into this category: erection loads; primary loads and secondary forces due to railway, highway and cycle traffic and pedestrian loads, flood action, short-term restraint actions due to temperature range and temperature gradient and the co-existent reactions due to subsoil.

(c) *Short-term dynamic actions*. Here we consider only railway loading, earthquake action and wind action, even though they can in many cases be simulated by equivalent static loads, obviating rigorous dynamic analysis.

In most codes, very little guidance is given on the simulation and effective application of actions to the bridge structures. Methods of simulation are dealt with in detail in the references given in the following subsections. The following guidelines are intended to cover cases of load application where most problems appear to arise.

4.4.1 Load Simulation Adapted to the Structural Configuration

Most bridges are analysed by plane-frame, plane-grid or space-frame methods with two- and three-dimensional finite element analyses reserved for special structures and the local analysis of complicated stress flow in a structural element where the plane section assumption does not hold. Most load simulation problems arise from the discretized nature of the skeletal structure that is used to simulate the bridge. A typical example is a twin-beam superstructure without transverse beams (see Fig. 4.4). A plane-grid simulation of this deck requires the connecting slab to be represented by a series of transversely spanning beam elements. It is normal to use from five to eleven such beam elements per span. The imposed gravitational loading acting on the superstructure DL and LL, if placed on the transverse members, will result in correct stress resultants

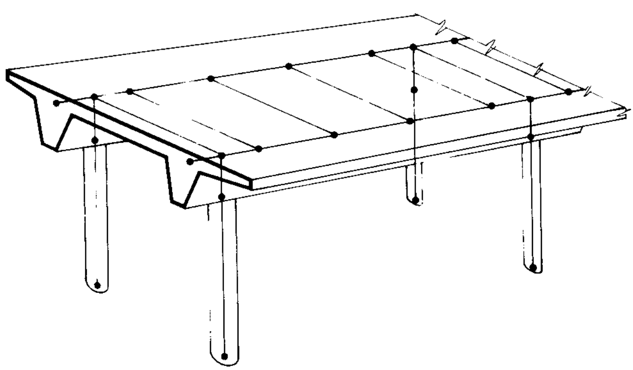


Figure 4.4 Twin-beam superstructure without crossbeams

for these members but incorrect sectional forces in the adjacent longitudinal beams. In fact, the results are only correct at the centre of each longitudinal beam element. This may be unacceptable in the case where such output is automatically coupled to design programs. A remedy to this dilemma would be to subject the longitudinal members to the continuously distributed load consisting of the equivalent end forces derived from the loads applied to the slab members.

However, in such a case, the stress resultants in the equivalent slab members are incomplete, since they only consist of the distributed load effects. The solution is to apply both forms of loading to the structure, each in a separate loadcase and always using the output from those members which have been loaded. The support reactions and sectional forces outside the loaded area are identical in both loadcases — thus resulting in a useful check for the correct evaluation of the equivalent loading.

In the case of genuine grid-type superstructures, it is advisable to resort to automatic load generator programs to generate the equivalent member loading due to traffic actions. The programs are usually coupled to an influence surface (or equivalent influence line) generator program for specified critical sections in the structure. In this case, the structure can be loaded in such a way as to result automatically in maximum and associated load effects at the predetermined sections for design purposes.

4.4.2 Equivalent Prestress Loading

Prestress loading is covered by Warner in Chapter 19 of Ref. 4.3, and Refs 4.6–4.9, 4.16,4.30,4.32 and 4.33. In modern design codes based on limit state design, the primary prestress effects and the secondary hyperstatic (or parasitic) prestress effects have to be factored differently in the design formulation $S_D \leq R_D$. This is required since the primary prestress effects form part of the resistance R_D of the section while the parasitic prestress effects form part of the action effects S_D (see Ref. 4.6, Vol. II, p. 76). It is therefore required to obtain the parasitic prestress effects separately from the primary effects in the analysis. In the normal prestress load simulation method,[4.8,4.16,4.32,4.33] the equivalent prestress load is evaluated by the process shown in Figs 4.5(a, b). The equivalent loading shown in Fig. 4.5(b) will result in stress resultants due to combined primary and parasitic effects. If, however, the primary prestress forces at *each member* end are added to the equivalent loading as shown in Fig. 4.5(c), the output will consist only of restraint (or parasitic) effects. A check for the correctness of the input is that the equivalent load of *each member* must be self-equilibrating while in Fig. 4.5(b) the sum of the equivalent loading on all members must be in equilibrium.

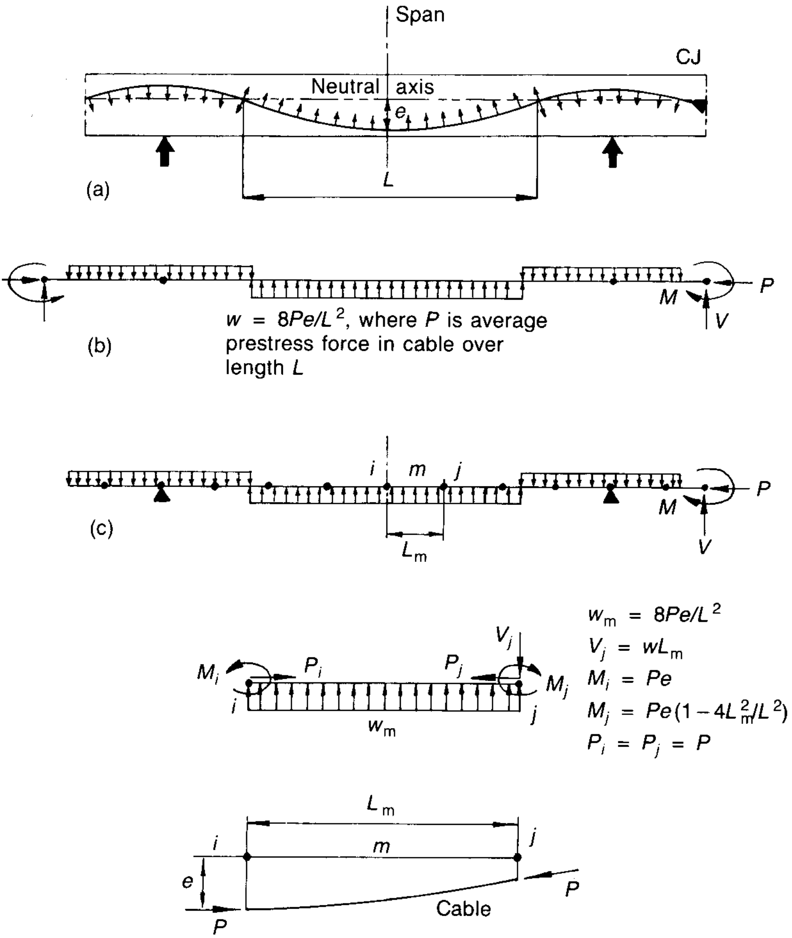


Figure 4.5 (a) Typical prestress profile showing forces exerted by prestress cable on the multispan beam; (b) equivalent prestress loading for primary–parasitic output; (c) equivalent prestress loading for parasitic output only

Another reason for keeping the primary and parasitic prestress effects separate, is the difference in their variation with time observed in stage-by-stage analysis of continuous structures. The primary prestress is only affected by the time-dependent losses while the parasitic prestress effects are affected by both time-dependent losses and the variation in relative structural stiffness with time. This phenomenon will be covered in more detail in Section 4.6.

4.4.3 Restraint Action Simulation

The restraint effects have been listed above in this subsection and reference should be made to applicable codes for their detailed definition (see Chapter 3, Section 3.4). For a more detailed description of these effects, refer to Chapter 11 of Ref. 4.3 and Refs 4.6–4.9 and 4.16. They all develop due to the inability of statically indeterminate structures to deform freely. The equivalent member loads required to produce these hyperstatic (parasitic) effects in the structural analysis are evaluated on the basis that they would produce the free deformation in each member when subjected to its equivalent load in isolation from the rest of the structure.

The equivalent fixed-end forces for any restraint action are therefore evaluated by reversing the free member end deformations and summing the forces developed at the fixed member ends. These equivalent forces are therefore dependent on the member stiffness, which in turn may be dependent on the member stresses and time. Realistic parasitic effects can only be obtained if the applicable Young's modulus and the effective section properties are used. It is imperative that the member stiffnesses used for the evaluation of the equivalent restraint loading be the same as those used for the subsequent analysis! If the results of the analyses are to be used for a serviceability limit state, it is usually sufficiently accurate to use member properties based on the nominal dimensions of the cross-sections and on the elastic short- or long-term moduli. However, if the results of the analysis are to be used in an ultimate limit state load combination, cognisance has to be taken of the effect of reduced member stiffness due to plasticity and cracking of concrete.

Methods of non-linear analysis for bridge structures are referred to in Section 4.5 and the application of non-linear analysis in Section 4.6.

4.4.4 Simulation of Wind Action (see Section 2.4.4)

Details of the evaluation of wind action on structures can be found in Refs 4.6,4.7,4.9,4.25,4.31 and 4.34. The evaluation of wind action on a bridge structure is extremely complex and several methods have been devised to simplify the load evaluation. The wind pressure consists of a steady-state pressure on which gust and eddy effects are superimposed. Besides the geographic, topographic and geometric influences on wind action, the natural frequency of the structure also plays a role on the fluctuating portion of the pressure. In practice, however, only wind-sensitive and/or large bridges need to be investigated for interaction with wind. Whether the pressure intensity is derived from empirical tables or by semi-deterministic wind–structure interaction, the result is a distributed member loading applied to the structure without any further simulation problems. Consideration must, however, be given to the eccentricity of the line of action relative to the centroidal member axis due to such effects as:

(a) shear centre and centroidal axis not coinciding;
(b) wind load on non-structural members such as balustrades;
(c) wind load on traffic.

The analytical procedures required for the evaluation of wind effects on bridge structures are discussed in Section 4.5 and the application of wind effects to bridge structures in Section 4.6.

4.4.5 Simulation of Earthquake Action

Details of the assessment of earthquake action on structures can be found in Chapter 15 (by Fintel and Ghosh) of Ref. 4.3 and in Refs 4.7,4.30 and 4.34–4.36. The structural response of a bridge structure depends on the seismic characteristics of a particular earthquake and on the stiffness characteristics (natural frequencies) of the structure. Equivalent static methods of evaluating nominal earthquake forces have been specified by certain codes applicable in areas where earthquake action is likely. In cases where structures are either exposed to higher-intensity earthquakes or where structures are more vulnerable to earthquake action, semi-dynamic simulation of these effects has been prescribed in the form of elastic and inelastic design spectra.

Multi-degree-of-freedom structures are best analysed by computer programs which cater for the above methods. In the case of very vulnerable structures, rigorous dynamic analysis is required using characteristic earthquake accelerograms. Specialized dynamic analysis computer programs are available which can perform elastic (linear) and non-linear analyses. The output consists of the structural response (displacements and stress resultants) for a series of time steps during the excitation phase. This can be very voluminous and has to be assessed with caution. Only the maximum effects are, however, required for design purposes. The analytical procedures and practical applications required are referred to in Sections 4.5 and 4.6, respectively.

4.5 Modern Analytical Methods Applicable to Bridges

A summary with brief descriptions of computerized methods used in modern bridge analysis is given here under the following subheadings:

1. Linear static stiffness analysis of skeletal structures.
2. Finite difference analysis.
3. Finite element analysis.
4. Finite strip analysis.
5. Dynamic stiffness analysis.
6. Non-linear stiffness analysis.

Some of these analytical methods have been covered in greater detail in Chapters 21 and 38 of Ref. 4.3.

4.5.1 Linear Static Stiffness Analysis of Skeletal Structures

A structrure can be discretized as a set of skeletal (one-dimensional) elements or members if it is considered that the second and third member dimensions play a negligible role in the solution (stress flow) of the structure when subjected to a particular set of loads. It must be recognized that the stiffness matrix of a one-dimensional (or beam) element is just a special case of a general finite element stiffness formulation. At the same time, the linear static portion of the matrix formulation is only a subset of the general non-linear dynamic finite equilibrium equation:[4.37]

$$\boldsymbol{M}^{t+\Delta t}\ddot{\boldsymbol{U}}^{(i)} + \boldsymbol{C}^{t+\Delta t}\dot{\boldsymbol{U}}^{(i)} + {}^{t}\boldsymbol{K}\Delta\boldsymbol{U}^{(i)} = {}^{t+\Delta t}\boldsymbol{R} - {}^{t+\Delta t}\boldsymbol{F}^{(i-1)} \quad [4.14]$$

where $i = 1, 2, 3 \ldots$ are iteration indices.

At time $t+\Delta t$,

$$^{t+\Delta t}\boldsymbol{U}^{(i)} = {}^{t+\Delta t}\boldsymbol{U}^{(i-1)} + \Delta\boldsymbol{U}^{(t)} \quad [4.15]$$

and

$\boldsymbol{M}$ = constant mass matrix
$\boldsymbol{C}$ = constant damping matrix
${}^{t}\boldsymbol{K}$ = tangent stiffness matrix at time t
${}^{t+\Delta t}\boldsymbol{R}$ = external nodal point load vector due to the body forces, surface loads and concentrated loads at time $t+\Delta t$
${}^{t+\Delta t}\boldsymbol{F}^{(i-1)}$ = nodal point force vector equivalent to the element stresses that correspond to the displacements ${}^{t+\Delta t}\boldsymbol{U}^{(i-1)}$
${}^{t+\Delta t}\boldsymbol{U}^{(i)}$ = vector of nodal point displacements at the end of iteration (i), and time $t+\Delta t$

The derivatives with respect to time are denoted by superior dots and the superscript (i) indicates an iteration index.

The linear static stiffness portion of this equation can be extracted as

$$\boldsymbol{KU} = \boldsymbol{R}$$

where $\boldsymbol{K}$ is the linear stiffness matrix, $\boldsymbol{U}$ the displacement vector and $\boldsymbol{R}$ the load vector due to joint loads and equivalent member loads (fixed end forces).

The basic steps involved in a linear static stiffness analysis are as follows:

(a) *Data preparation*. This is done by an engineer and/or a generation program:
  1. Line diagram. Decide on the type of skeletal modelling.
  2. Numbering of joints and members and deciding on support conditions.

3. Member properties are evaluated and member end connectivities defined.
4. Loads are evaluated and applied to joints and members.

(b) *Calculations*. Ten stages are involved:

1. Degree-of-freedom table is generated.
2. Member location table is generated.
3. Member direction cosines are calculated.
4. Member *transformed* stiffness matrices, $\boldsymbol{k}_{XYZ}$, are calculated.
5. Main structure stiffness matrix, $\boldsymbol{K}_{XYZ}$, is assembled using 1, 2 and 4, above.
6. Fixed end reactions are evaluated for all loaded members $\boldsymbol{f}_{XYZ}$.
7. Column matrix of joint loads, $\boldsymbol{R}_{XYZ}$, is assembled.
8. Deformation matrix, $\boldsymbol{U}_{XYZ}$, is solved for using $\boldsymbol{KU} = \boldsymbol{R}$.
9. Appropriate joint deformations are back-substituted into the transformed member stiffness equilibrium equation:

   $$\boldsymbol{p}_{XYZ} = \boldsymbol{k}_{XYZ}\boldsymbol{U}_{XYZ} + \boldsymbol{f}_{XYZ}$$

   to yield the global axis stress resultants $\boldsymbol{p}_{XYZ}$.
10. Stress resultants in local (or member) axis system are evaluated from $\boldsymbol{p}_{xyz} = \boldsymbol{T}\boldsymbol{p}_{XYZ}$, where $\boldsymbol{T}$ is the transformation matrix.

    XYZ here refers to the global or structure axis system, *xyz* to the local or member axis system, and the bold italic type denotes a matrix or tensor.

The above basic steps are required in all static stiffness solution techniques, varying only in the degree of complexity of the particular discretization and solution scheme.

4.5.2 Finite Difference Analysis

Finite difference mesh solution procedures have been applied successfully to various structural analysis problems. The traditional difference approach with its boundary formulation problems was found to be very limited in its range of applicability. However, bridge decks which could be simulated by orthotropic plate theory have been successfully and economically solved by this approach. Some finite strip formulations employ finite differences in the transverse direction.[4.38] In some modern applications of finite difference formulations, the final product (solution procedure) is virtually indistinguishable from procedures based on finite element formulations.[4.39] Certain of these techniques are also used in dynamic analysis, dealt with in Section 4.5.5. Methods based on the finite difference technique are covered in Refs 4.39 and 4.40.

4.5.3 Finite Element Analysis

The finite element method of stress analysis has been dealt with in Chapter 21, Section 3 of ref. 4.3 and by Cheung in Chapter 38. Further information can be found in Refs 4.4, 4.19–4.23, 4.37–4.58. This is a form of structural analysis which is developing at a fast rate, resulting in rapid improvement of elements, solution techniques and analytical procedure. It is therefore advisable to refer to the latest developments in relevant literature, such as the *International Journal for Numerical Methods in Engineering* and *Computers and Structures*, published by John Wiley & Sons, New York, and Pergamon Press, Oxford, respectively.

The general finite element equilibrium equation 4.14 can form the basis of most spatial and temporal finite element discretization formulations.

4.5.4 Finite Strip Analysis

The finite strip method of analysis has been dealt with by Cheung in Chapter 21, Section 4 and in Chapter 38 of Ref. 4.3 and differs from the standard finite element method of analysis in that it uses polynomials as the displacement functions in some directions and continuously differentiable smooth series in other directions with the stipulation that such series functions should satisfy the boundary conditions at the ends of the strip. Further information can be obtained from Refs 4.22, 4.23, 4.38, 4.40, 4.41, 4.46 and 4.48. The finite strip method lends itself to the analysis of bridge structures due to the prismatic shape and elongated form of the superstructure. Relative to the finite element method of analysis, this method is less general in application but more economic on computer time. Many different formulations have been used in the form of infinite series over the length of the structure and displacement functions transverse to the superstructure. As for the finite element methods, the finite strip formulations have also been derived from basic variational principles of mechanics. One interesting aspect of the finite strip method is that it lends itself to the substructuring of matrices with further reduction in computational effort. This method has been successfully applied to straight, skew and curved-plate and folded-plate structures.

4.5.5 Dynamic Stiffness Analysis

Chapter 15 (by Fintel and Ghosh) of Ref. 4.3 deals with the dynamic analysis, with particular reference to the action of earthquakes. For further information on dynamic analysis methods, see Refs 4.37, 4.50, 4.52 and 4.57.

The finite element equilibrium equation (Eq. 4.14) can form the basis for this method of analysis. In the absence of non-linearity, it can be rewritten as

$$\boldsymbol{M}{}^t\ddot{\boldsymbol{U}} + \boldsymbol{C}{}^t\dot{\boldsymbol{U}} + \boldsymbol{K}{}^t\boldsymbol{U} = {}^t\boldsymbol{R} \qquad [4.16]$$

where

$\boldsymbol{M}$ = constant mass matrix
$\boldsymbol{C}$ = constant damping matrix
$\boldsymbol{K}$ = linear stiffness matrix
${}^t\boldsymbol{R}$ = time-dependent local vector
${}^t\boldsymbol{U}$ = time-dependent displacement vector
${}^t\dot{\boldsymbol{U}}$ = time-dependent velocity vector
${}^t\ddot{\boldsymbol{U}}$ = time-dependent acceleration vector

There are a number of time integration schemes employing mode superposition which give satisfactory solutions of a discretized system of equations defining a structure based on Eq. 4.16. These can be found in Refs 4.37, 4.50, 4.52 and 4.57.

In the case of bridges, dynamic stiffness methods are mainly used for earthquake analysis. Earthquake actions are usually applied in the form of base excitation expressed as an acceleration time function. Analysis for earthquake action has been dealt with in greater detail by Fintel and Ghosh in Chapter 15 of Ref. 4.3. The application of earthquakes to bridge structures is briefly summarized in Section 4.6.

4.5.6 Non-linear Stiffness Analysis

Non-linear stiffness analysis can be separated into time-dependent and static formulations. Furthermore, one must distinguish between material and geometric non-linearity although both phenomena are often dealt with in the same analysis since the deformations due to material plasticity often give rise to large displacements which require updating of the structure geometry in order to take account of the '$P-\Delta$' effects. In the case of time-independent stiffness analysis, Eq. 4.14 can be rewritten as

$${}^t\boldsymbol{K}\,\Delta\boldsymbol{U}^{(i)} = {}^t\boldsymbol{R} - \boldsymbol{F}^{(i-1)} \quad i = 1, 2, 3 \ldots \qquad [4.17]$$

where

$$\Delta\boldsymbol{U}^{(i)} = \boldsymbol{U}^{(i)} - \boldsymbol{U}^{(i-1)}$$

and

${}^t\boldsymbol{K}$ = tangent stiffness matrix where ^{t} indicates the load path incremental or sequences of loading only in time-independent problems
${}^t\boldsymbol{R}$ = external nodal point load vector

${}^{t}\boldsymbol{F}^{(i-1)}$ = nodal point force vector equivalent to the element stresses that correspond to the displacements $\boldsymbol{U}^{(i-1)}$

$\boldsymbol{U}^{(i)}$ = vector of nodal point displacements at the end of iteration (i)

Equilibrium iteration ($i = 1, 2, 3 \ldots$) is often necessary in this kind of analysis in order to ensure convergence of load incrementation techniques.

Time-dependent non-linear stiffness analysis can be represented in the form of the equilibrium equation 4.14. The load vector $\boldsymbol{R}$ and the stiffness matrix $\boldsymbol{K}$ can be time-dependent and a solution at specified time intervals is required in order to determine maximum structural response during the interval of time-dependent loading.

A form of time-dependence in bridge structures is encountered in stage-by-stage analysis involving time-dependent material properties. Since inertial effects do not play a role in this type of analysis, no dynamic formulation is required. Therefore, this type of analysis is usually performed in a static mode with stepwise varying properties. A typical application of this type of analysis is dealt with in Section 4.6.

The basis and advanced development of non-linear stiffness analysis theory and methods of analysis can be obtained from specialist publications.[4.4,4.49–4.54]

4.6 Computer Applications to Bridge Analysis

The methods of analysis described in Ref. 4.3 apply equally to bridge structures. The differences which do occur are not in the analytical theory, but in the application thereof. This will be illustrated by considering a few examples of action effects on bridge structures as found in the evaluation of earthquake effects, wind effects, the long-term effects of stage construction and the assessment of non-linear effects in bridge structures.

4.6.1 The Application of Earthquake Action to Bridge Structures

In Chapter 15 of Ref. 4.3 Fintel and Ghosh describe the nature of earthquake action on structures in general and the methods applied to evaluate the structural response thereto. In Section 4.4 the simulation of these effects on bridge structures is summarized and in Section 4.5 the basis of modern dynamic analysis is given. References 4.7, 4.30 and 4.34–4.36 deal with particular aspects that are relevant to bridges. A method of evaluation that is suitable for application to concrete bridges is summarized below.

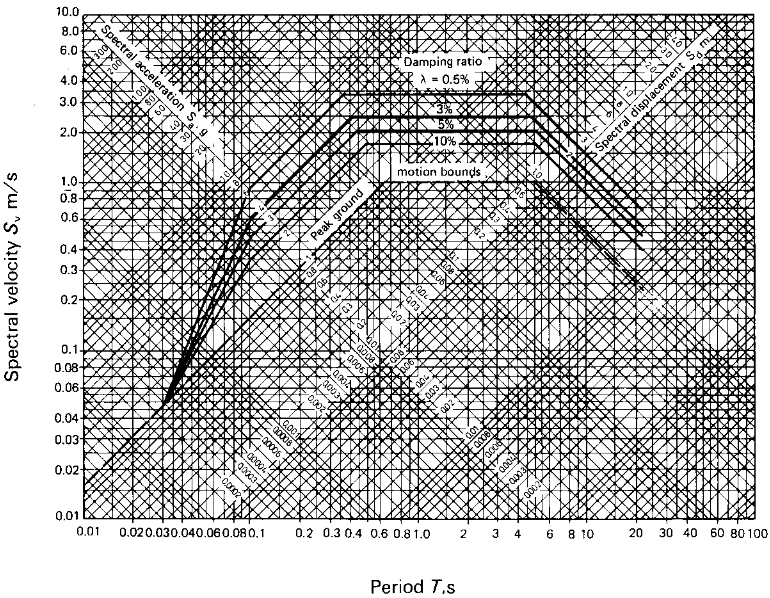


Figure 4.6 The elastic average response spectrum bounds, normalized to a ground acceleration of $1.0\,g$

Table 4.1

Stress level	Type and condition of structure	Damping ratio
Working stress no more than about half yield	(a) Welded steel, prestressed concrete, well-reinforced concrete (only slight cracking)	0.02
	(b) Reinforced concrete with considerable cracking	0.03–0.05
	(c) Bolted and/or riveted steel	0.05–0.07
At or just below yield point	(a) Welded steel, prestressed concrete under full prestress (slight cracking)	0.05
	(b) Prestressed concrete in cracked state	0.07
	(c) Reinforced concrete	0.07–0.10
	(d) Bolted and/or riveted steel	0.10–0.15

4.6.1.1 Outline of Method

The method is based on the average response spectrum of structures, which was derived from the response of single-degree-of-freedom structures when subjected to a range of recorded earthquakes. The elastic average response spectrum bounds are shown in Fig. 4.6 normalized to a ground acceleration of $1.0g$. For single-degree-of-freedom structures, the design spectrum is entered with the natural period of the structure. The maximum acceleration, S_a, is then read off at the applicable damping ratio. Typical damping ratios are listed in Table 4.1. This can be used as a manual method on small structures or as a computerized method on larger structures. In a multi-degree-of-freedom system that has independent or uncoupled modes of deformation, each mode responds to the excitation as an independent single-degree-of-freedom system. Generally, only the response in the fundamental natural frequency is required in the orthogonal orientations of the bridge structure. The equivalent static load to which the bridge is subjected can then be calculated from

$$F_x = k_{gh} I f S_a m_x \qquad [4.18]$$

where

k_{gh} = peak horizontal ground acceleration (see Table 4.2)
I = importance factor of the structure which generally lies in the range 1.0–1.3
f = foundation factors which are given in Table 4.3
S_a = seismic response factor for the structure read from the response spectrum in Fig. 4.6
m_x = the mass of the structure or part of the structure considered

The dynamic earthquake loading is in this way reduced to an equivalent static loading.

4.6.1.2 Adjustment for Elastic–Plastic Behaviour

For a structure designed to undergo elastic–plastic deformation under

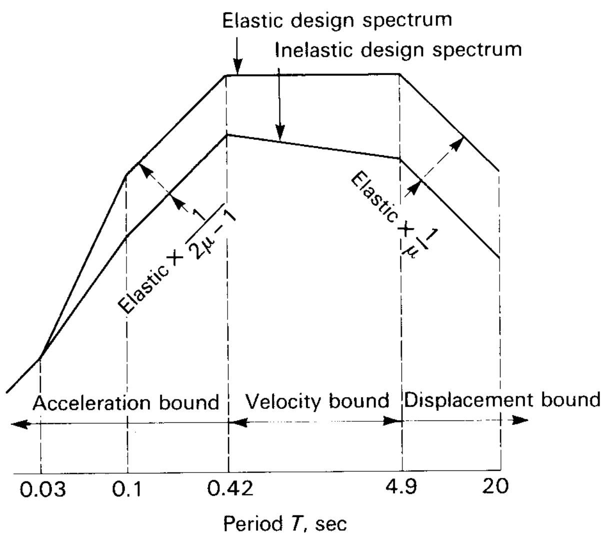


Figure 4.7 The transformation from elastic to elastic–plastic design spectrum. Note that the corner period values are unchanged

Table 4.2

Modified Mercalli intensity at epicentre, MM	Maximum ground acceleration, k_{gh}, at epicentre
ii–iii	$0.003g$
iv–v	$0.01g$
vi	$0.03g$
vii–viii	$0.1g$
ix	$0.3g$
x–xi	$1.0g$

Table 4.3

Type and depth of soil	f
Rock, dense and very dense coarse-grained soils, very stiff and hard fine-grained soils; compact coarse-grained soils and firm and stiff fine-grained soils from 0 to 15 m deep	1.0
Compact coarse-grained soils, firm and stiff fine-grained soils with a depth greater than 15 m; very loose and loose coarse-grained soils and very soft and soft fine-grained soils from 0 to 15 m deep	1.3
Very loose and loose coarse-grained soils and very soft fine-grained soils with depth greater than 15 m	1.5

earthquake action, the average elastic response spectrum may be modified as follows in order to obtain the inelastic design spectrum.

The elastic spectrum, for any given damping ratio, is modified along the displacement bound region by multiplying the bound line by a factor $1/\mu$, and along the acceleration bound region by multiplying the bound line by a factor $1/\sqrt{(2\mu-1)}$.

A typical conversion is shown in Fig. 4.7. Note that the period at the corner points is unchanged. The ductility factor μ is defined as the total elastic–plastic deformation of the structure divided by its total elastic deformation at yield.

Table 4.4 gives values of μ for typical structural configurations.

The maximum acceleration, S_a, is then read off at the modified spectrum bound and used in Eq. 4.18 as before. The equivalent static analysis can be performed by loading the structure with the forces F_x. The sectional forces obtained are the true elastic–plastic response of the structure. The elastic–plastic displacements can be obtained from the analysis by multiplying the displacements by the ductility factor μ.

The maximum structural ductility factor μ given in Table 4.4 must be used, except where tests and calculations demonstrate that higher values are justified.

For structural ductility factors μ equal to or greater than 5, this

Table 4.4

Case	Type or arrangement of resisting elements	Structural ductility factor, μ
1	Structural steel unbraced bridges, piers and superstructures adequately designed to resist the total lateral forces by bending of the members in accordance with their relative rigidities considering the interaction of the various parts	4
2	Structures with braced flexural piers of structural steel, or substructures consisting of slender reinforced concrete columns and superstructures in braced structural steel, or in reinforced or prestressed concrete, adequately designed to resist the total lateral force in accordance with their relative rigidities considering the interaction of the various parts	3
3	Structures with piers or abutments of shear wall proportions and superstructures in monolithic reinforced or prestressed concrete adequately designed to resist the total lateral force in accordance with their relative rigidities considering the interaction of the various parts	2

recommended procedure should not be used, but a time series analysis with realistic material behaviour should be performed. It should be noted that, in order to achieve a given structural ductility, the ductility factors of some individual members, particularly those of girders, may need to be greater than the overall structural ductility.

4.6.1.3 Application to Multi-Degree-of-Freedom Structures

For symmetrical structures, the influence of seismic disturbances is to be considered along both principal axes of symmetry. For non-symmetrical structures, seismic effects may be considered along two arbitrarily chosen orthogonal axes. For both symmetrical and non-symmetrical structures, the seismic effects along the two orthogonal axes may be considered independently of one another. Where it is deemed necessary to consider the influence of vertical seismic motions, the average vertical design spectrum may be taken as two-thirds the average horizontal seismic response spectrum (Fig. 4.6). It should be noted that the relevant resonance frequencies of the structure and its components in the vertical direction are generally quite different from those in the horizontal direction. The response caused by vertical excitation is to be combined with the response caused by horizontal excitation as specified below.

The structure is first analysed to determine its natural modes of vibration and the corresponding natural frequencies (or periods). This generally involves the solution of an eigenvalue formulation of the structure of the form

$$\left((m) - \left[\frac{1}{\omega^2}\right](K)\right)\{x\} = 0$$

where

(m) = diagonal matrix of lumped masses representative of the dead and superimposed dead load imposed on the structure
(K) = stiffness matrix of the structure
$\{x\}$ = general displacement vector of all degrees of freedom of the structure
$[1/\omega^2]$ = inverse square vector of the natural radial frequencies ω

Computation of structural response Each mode is first assumed to behave independently in the earthquake response. The structural response acceleration S_{ai} (in units of gravitational acceleration) can thus be evaluated independently for each mode i with period T_i in the way shown in Sections 4.6.1.1 and 4.6.1.2. Each mode will participate in the total combined response by an amount $\gamma_i \Psi_{ij} S_{ai}$. The 'modal participation factor' γ_i is evaluated for the ith mode as follows:

$$\gamma_i = \frac{\Sigma\, \bar{m}_j \Psi_{ij}}{\Sigma\, m_j\, (\Psi_{ij})^2}$$

where

$\Sigma \equiv \Sigma_{i=1}^{n}$
m_j = lumped mass free to move in the direction of the degree of freedom j
$\bar{m}_j$ = $m_j \cos \alpha_j$ where α_j is the angle between the modal plane of movement of the mass m_j and the direction of excitation of the earthquake
Ψ_{ij} = normalized mode component of the ith mode in the direction of the jth degree of freedom
n = number of degrees of freedom of the structure

The generalized set of equivalent static forces acting on the structure can thus be written as:

$$F_{ij} = k_{gh}\, If \gamma_i\, \Psi_{ij}\, S_{ai}\, m_j.$$

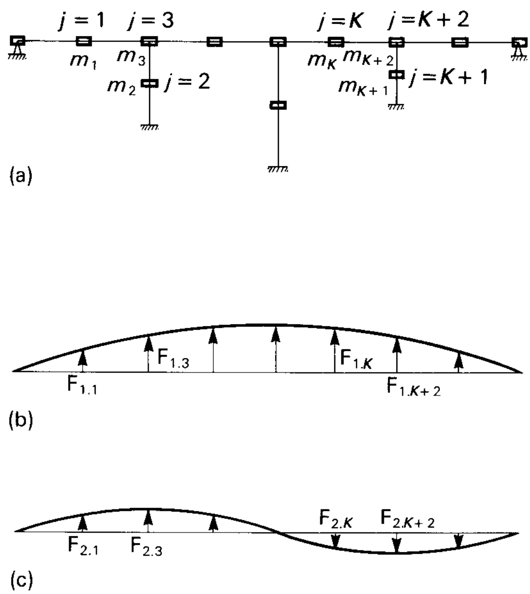
4.6.1.4 Combination of Modes

By the process described in Section 4.6.1.2, the maximum response of each mode of the multi-degree-of-freedom structure can be found. The maximum values of combined effects are obtained by taking the square root of the sum of the squares of the effects from each mode. The first three natural modes of vibration of the structure are normally sufficient to obtain maximum combined effects.

4.6.2 The Application of Wind Action on Bridge Structures

The nature of wind action and its simulation, referred to in Chapter 2, Section 2.4.4, is described in Sections 4.4 and 4.5 of this chapter, and can be studied in more detail in Refs 4.7, 4.9, 4.25, 4.31, 4.34 and 4.59–4.61. The effect of wind on bridge structures differs from that on tall buldings, for which most of the field research has been done. The methods of analysis based on this work are consequently not directly applicable to bridges. Most concrete bridge structures are, however, not very sensitive to the gustiness of wind because of the stiffness (short natural periods of vibration) or size (low cross-correlation of gusts) of bridge members (see Chapter 2, Section 2.4). Realistic equivalent static wind loads can therefore be evaluated with sufficient accuracy for most types of bridge structure. The application of wind loads to bridge structures has been dealt with in great detail in the codes referred to in Refs 4.7 and 4.9. Some large concrete bridge structures, as well as slender ones, are susceptible to wind gusting effects and should be analysed taking account of their natural frequencies and corresponding gust factors as described in Ref. 4.25 or 4.34. The natural frequency or period of the bridge structure or of its slender component members requires evaluation. This is best done by an eigenvalue solution performed on a computer. The structural discretization and mass distribution of members is the same as that required for dynamic earthquake analysis, as illustrated in Fig. 4.8.

Figure 4.8 (a) Idealized bridge structure for transverse excitation; (b) forces due to first transverse mode $i = 1$; (c) forces due to second transverse mode $i = 2$



4.6.3 Stage-by-stage Construction Analysis

A summation of the load effects at all construction stages of a bridge evaluated by elastic analysis does not produce the correct final forces or displacements, because of the tendency of 'green' concrete to undergo creep deformation during construction. The exact solution of the redistribution of stresses due to creep effects requires considerable computational effort. There are various inherent inaccuracies in the assumptions on which the creep effects are based; the methods applied should therefore be of a comparable order of accuracy so as to reduce the work to a realistic level. The two effects to be evaluated are: (a) redistribution of sectional forces; and (b) member (or nodal) deflections. While the effects under (a) are only dependent on the relative stiffness of members (and changes thereof with time), those under (b) are dependent on the absolute or total stiffness of the structure over a period of time. It is therefore often necessary, for slender bridge structures, to use more accurate methods of analysis to obtain member deflections than are required to evaluate changes in member forces.

The phenomenon of creep is defined by the relevant codes such as Refs 4.6 and 4.9, and the methods of evaluating creep effects have been widely dealt with in technical publications.[4.62–4.66] The following is a summary of the effects.

When a structure is constructed in 'one go', strains and deformations due to creep (and shrinkage) are proportional to the corresponding elastic effects. No tendency therefore exists for the redistribution of forces since no restraints are built up. This does not, however, apply to a structure built in stages. Here, two considerations are important:

(a) The sum of the elastic effects during stage construction is not equal to the effects from a 'one go' (or built-in-one) structure. The differences between the two stages are reduced by creep, i.e. the structural forces tend to redistribute from the 'sum of stages' to the 'one go' state (see Fig. 4.9).
(b) Since at any particular stage of construction concretes with different

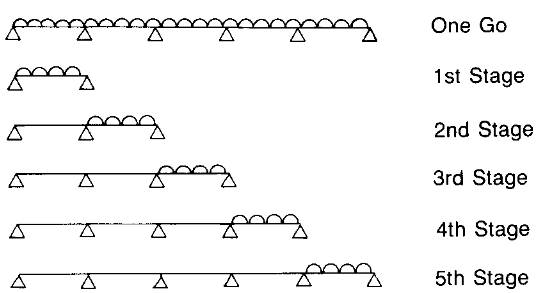


Figure 4.9 Stage-by-stage construction

creep characteristics occur in different parts of the structure due to the differences in age and time under load, the creep deformations are not directly proportional to their respective elastic deformations. This results in creep redistribution. The younger concrete undergoing larger creep deformation transfers (or sheds) internal forces (stresses) to older concrete undergoing less creep deformation. If the creep which takes place during the construction period is small relative to the creep which takes place after completion, then a simplified approach can be applied:

$$M_F^t = \Sigma M_s^i - (\Sigma M_s^t - M_{OG}^i)R \qquad [4.19]$$

where

M_F^t = final moment at point i
ΣM_s^t = sum of all stage moments at point i evaluated by elastic analysis
M_{OG}^t = moment at point i due to a 'one go' analysis using elastic (short-term) or long-term member properties
R = redistribution factor
= $\phi/(1+\eta\phi)$

and

ϕ = average total creep factor of the concrete members
η = relaxation coefficient, which can be taken as 0.85

This operation can easily be programmed by using appropriate scale factors on the computer output of the respective load cases. If, however, the creep effects during the construction period cannot be neglected, then the following procedure should be followed.

Using the notation ${}^{t_r}M_{m_i}^n$ to describe moment at the position m due to load case i at time t_r on the structural system n, then for the load case $i = 1$:

Construction stage $n = 1$:

$${}^{t_1}M_{m_1}^1 = {}^{t_0}M_{m_1}^1 \qquad [4.20]$$

where t_0 is the time of loading of load case $i = t_1$, and t_1 is the time at start of first creep interval t_1 to t_2.

Construction stage $n = 2$:
Start:

$${}^{t_1}M_{m_1}^2 = {}^{t_1}M_{m_1}^1 \qquad [4.21]$$

End:

$${}^{t_2}M_{m_1}^2 = {}^{t_1}M_{m_1}^1 - ({}^{t_1}M_{m_1}^1 - M_{m_1}^{2*})R_{1.2} \qquad [4.22]$$

where

$$R_{1.2} = \frac{\Delta\phi}{1+\eta\Delta\phi}$$

$$\begin{aligned}\Delta\phi &= \text{change of } \phi \text{ factors in the time interval } t_2-t_1\\ &= \phi_{t_2}-\phi_{t_1}\\ \eta &= 0.85\end{aligned}$$

and * denotes an analysis performed with a different effective Young's modulus ${}^{t_2}E = E/(1+\phi_{t_2})$ for each member.
At this stage, $\phi_t = \phi_{t_2}$.

Construction stage $n = 3$:
Start:

$$^{t_2}M^3_{m_1} = {}^{t_2}M^2_{m_1} \qquad [4.23]$$

End:

$$^{t_3}M^3_{m_1} = {}^{t_2}M^2_{m_1} - ({}^{t_2}M^2_{m_1} - M^{3*}_{m_1})R_{2.3} \qquad [4.24]$$

$$R_{2.3} = \frac{\Delta\phi}{1+\eta\Delta\phi}$$

where

$$\begin{aligned}\Delta\phi &= \phi_{t_3}-\phi_{t_2}\\ \eta &= 0.85\\ {}^{t_3}E &= E/(1+\phi_{t_3}) \text{ for each member in analysis*}\end{aligned}$$

In general, one can write the expression for load case i at time t_r and construction stage n:

$$^{t_r}M^n_{m_1} = {}^{t_{r-1}}M^{n-1}_{m_i} - ({}^{t_{r-1}}M^{n-1}_{m_i} - M^{n*}_{m_i})R_{n-1,n} \ldots \qquad [4.25]$$

where

$$R_{n-1,n} = \frac{\Delta\phi}{1+\eta\Delta\phi}$$

$$\begin{aligned}\Delta\phi &= \phi_{t_r}-\phi_{t_{r-1}}\\ \eta &= 0.85\\ {}^{t_r}E &= E/(1+\phi_{t_r}) \text{ for each member in analysis*}\end{aligned}$$

This operation can also be computerized using an appropriate summation algorithm to combine the respective output files of the computer analyses.

Care must be taken if the displacements are required from such an analysis. Since this is a stepwise non-linear analysis, the principle of superposition does not hold for displacements. They are best evaluated separately from first principles using the appropriate sectional forces and long-term sectional properties.

4.6.4 Non-linear Effects in Bridge Structures

The improvements in methods of analysis and design of structures have increased the accuracy of assessment of load effects and strength capacities. This has brought about a tendency towards the design of very slender bridges, with the result that certain effects, like non-linear structural behaviour, that could previously be neglected may now be as important as some of the primary load effects. In concrete bridge structures, these non-linear effects are usually negligible at service load level. Since it is accepted practice to perform linear elastic analysis at *service* load level and then combine the results with appropriate load factors to obtain ultimate load combination effects, this can result in faulty *ultimate* load capacity estimates. The assessment of the non-linear effects has been subject to much investigation and development. The state-of-the-art has now reached a level where most of the required effects can be assessed with sufficient accuracy for the purposes of the design engineer. Certain design codes provide the designer with information in order to assess non-linear effects.[4.6,4.9,4.10] More information can be obtained from technical publications.[4.14,4.67–4.78]

Depending on the accuracy required, it may be necessary to ensure that the selected non-linear method of solution allows for some or all of the following effects:

(a) creep in concrete including the effects of mixed short- and long-term loads;
(b) shrinkage in concrete;
(c) temperature effects at any level of stress in the concrete and steel;
(d) presence of cracks in concrete;
(e) partial foundation fixity;
(f) effects of initially built-in forces (e.g. prestress);
(g) biaxial bending;
(h) accurate simulation of cyclic load effects in concrete.

In addition, the analysis may have to allow for combined (interactive) material and geometric non-linearity. The specification of members should allow for arbitrary shapes including unsymmetric box-type cross-sections with arbitrary arrangements of reinforcement. Incremental loading should be allowed for with selected output options at any stage of loading up to failure or instability.

The stress–strain relation and its effect on the simulation of member stiffness under variable loading play an important part in (a), (b), (c), (d), (f), (g) and (h) above, and the selected program should be tested for all the above effects before using it for actual bridge analysis.

In bridge structures it is often possible to isolate a non-linear portion of the structure and analyse it separately in more detail. The procedure to be followed can take the following form:

(a) Analyse the bridge as a whole for a particular loading configuration (i.e. dead load, prestress, live load, temperature, shrinkage, etc.) using a linear analysis.
(b) Isolate the portion of the structure which is likely to behave non-linearly (e.g. a column assembly).
(c) Subdivide the isolated portion into sufficient elements so that it can reproduce the non-linear effects.
(d) Subject the isolated portion to incremental loading using a non-linear program.

Note: It is important to impose the correct forces on the boundary planes of the isolated portion of the structure at each loading increment. The boundary planes must therefore be chosen at positions where the stress resultants are small and vary approximately linearly in proportion to the loading. The results of such an analysis can be sufficient for design purposes if care is taken in the choice of the boundaries of isolation.

A related procedure called the model-column method is described in Section 14.4.3.2 of the CEB–FIP code.[4.6]

4.7 Model Testing

Model testing of structures in general is described by White in Chapter 23 of Ref. 4.3. The methods and procedures covered therein are equally applicable to bridge structures, but a few aspects that are peculiar to bridge structures require further mention. Whereas building structures are usually clad with materials and finishes that can have a significant effect on the structural behaviour, this rarely applies to bridges. Furthermore, because of its elongated form, it can usually be simulated more readily. The type of model and its detailing depends on the problem being investigated. Parts only of the structure need be modelled in some cases.

In addition to using the materials and procedures described by White,[4.3] various tests[4.79–4.84] have in recent years been done on large reinforced and prestressed concrete models of bridges up to a scale of 1:2.82. Such tests can accordingly give a very accurate simulation of the whole process of loading, from construction at various stages up to completion and thereafter.

Wind-tunnel tests may be essential in the case of certain long-span cable-stayed bridges. A full model is usually scaled down to 1:200 or higher and sectional models in the range 1:30 to 1:50.

References

4.1 Timoshenko S P 1953 *History of Strength of Materials* McGraw-Hill, New York, 452 pp

4.2 Cope R J, Bungey J H Apr 1975 Examples of computer usage in bridge design *J. Struct. Div. Am. Soc. Civ. Eng.* **101** (ST4): 779–93

4.3 Kong F K, Evans R H, Cohen E, Roll F (eds) 1983 *Handbook of Structural Concrete* Pitman, London

4.4 Bathe K J, Ramaswamy S 1979 On three-dimensional non-linear analysis of concrete structures *Nucl. Eng. & Design* **52**: 385–409

4.5 ASCE 1982 *State-of-the-Art Report on Finite Element Analysis of Reinforced Concrete* p 210

4.6 Comité Euro-International du Béton (CEB) 1978 *International System of Unified Standard Codes of Practice for Structures* vol I: *Common Unified Rules for Different Types of Construction and Material* 123 pp, and vol II: *CEB–FIP Model Code for Concrete Structures* Bulletin d'information, N.124/125-E, 348 pp

4.7 National Institute for Transport and Road Research 1981 *Technical Methods for Highways: Code of Practice for the Design of Highway Bridges and Culverts in South Africa* TMH 7 — Parts 1 and 2, Pretoria, 100 pp

4.8 Pennels E 1978 *Concrete Bridge Designer's Manual* Cement and Concrete Association, London 164 pp

4.9 British Standards Institution *BS 5400: Parts 1–10: 1978–1984* Steel concrete and composite bridges

4.10 American Concrete Institute, 1981 *ACI Manual of Concrete Practice* Part 4, Detroit, Chapter 3

4.11 Parrot L J Oct 1979 *Simplified Methods of Predicting the Deformation of Structural Concrete* Cement and Concrete Association, London, Development report 3, 11 pp

4.12 Vlasov V Z 1961 *Thin Walled Elastic Beams* 2nd edn, translated from Russian by Israel Programme for Scientific Translations, Jerusalem, p 355

4.13 Zbirohowski-Koscia K 1967 *Thin Walled Beams from Theory to Practice* Crosby Lockwood, London p 53

4.14 Bathe K J, Ramaswamy S 1979 On three-dimensional non-linear analysis of concrete structures *Nucl. Eng. and Design* **52**: 385–409

4.15 Hobbs D W Nov 1979 *Shrinkage-Induced Curvature of Reinforced Concrete Members* Cement and Concrete Association, London, Nov 1979, Development report 4, 19 pp

4.16 Hambly E C 1970 *Bridge Deck Behaviour* Chapman and Hall, London, pp 187–203

4.17 Meredith D, Witmer E A Jun 1981 Non-linear theory of general thin-walled beams *Comput. Struct.* **13**: 3–9

4.18 Dittmann G, Bondre K G 1979 Die neue Rheinbrücke Düsseldorf-Flehe/Neuss-Uedesheim: statishce Berechnung des Gesamtsystems *Bauingenieur* **54**: 59–66

4.19 Douglas M R, Parekh C J, Zienkiewicz O C 1971 Finite element programs for slab bridge design *Symposium on Concrete Bridge Design* vol I, American Concrete Institute, SP-26, pp 117–141

4.20 Davies J D, Parekh C J, Zienkiewicz O C 1971 Analysis of slabs with edge beams *Symposium on Concrete Bridge Design* vol I, American Concrete Institute, SP-26, pp 142–165

4.21 Dravid P S, Mehta C L 1971 Bridge grids with integral slabs — a new analytical approach *Symposium on Concrete Bridge Design* vol I, American Concrete Institute, SP-26, pp 166–81

4.22 Cheung Y-K Orthotropic right bridges by the finite strip method *Symposium on Concrete Bridge Design* vol I, American Concrete Institute, SP-26, pp 182–205

4.23 Cheung Y-K 1971 Analysis of box girder bridges by the finite strip method *Symposium on Concrete Bridge Design* vol I, American Concrete Institute, SP-26, pp 357–78

4.24 Kollbrunner C F, Basler K 1966 *Torsion*, Springer Verlag, New York, 280 pp (in German)

4.25 The modern design of wind-sensitive structures, *Proc. Seminar of Institution of Civil Engineers* London, 18 June 1970, Construction Industry Research and Information Association, London, 1971, 139 pp

4.26 Rajagopalan K S 1969 Comparison of loads around the world for the design of highway bridges *Symposium on Concrete Bridge Design* American Concrete Institute, SP-26, vol 1, pp 35–48

4.27 Liebenberg A C Mar 1974 *Report to the National Transport Commission of the Republic of South Africa on Proposals for a Uniform Specification of Live Loading due to Traffic on National Road Bridges* Liebenberg & Stander, Cape Town pp 9–217

4.28 Niyogi P K, Shah H C, Doshi K D, Tang Wilson 1971 Statistical evaluation of load factors for concrete bridge design *Symposium on Concrete Bridge Design* vol I, American Concrete Institute, SP-26, pp 75–111

4.29 Nayak G C, Davies J D 1971 Influence characteristics for slab bridges *Symposium on Concrete Bridge Design* vol I, American Concrete Institute, SP-26, pp 75–111

4.30 FIP Nov 1977 *Recommendations for the Design of Aseismic Prestressed Concrete Structures* Cement and Concrete Association, London 28 pp

4.31 Sigbjornsson R, Hjorth-Hansen E Jan 1981 Along-wind response of suspension bridges with special reference to stiffening by horizontal cables *Eng. Struct.* **3**: 27–37

4.32 Pfaffinger D D 1979 Prestressing load cases in linear and non-linear finite element analysis *Proc. ADINA Conf.* Cambridge, Mass., Aug 1979, pp 363–82

4.33 Sawko F, Wilcock, B K 1971 Automatic design of pre-stressed concrete bridges by electronic computer *Symposium on Concrete Bridge Design* vol II, American Concrete Institute, SP-26 pp 690–713

4.34 Associate Committee on the National Building Code *National Building Code of Canada* NRCC 17303, 1980, and supplement NRCC 17724, National Research Council of Canada, Ottawa, 1980, 547 and 293 pp respectively

4.35 Blume, J A, Newmark N M, Corning L H 1961 *Design of Multistorey Reinforced Concrete Buldings for Earthquake Motions* Portland Cement

Association, Skokie, 318 pp
4.36 Newmark N M, Rosenblueth E 1971 *Fundamentals of Earthquake Engineering* Prentice-Hall, Englewood Cliffs, NJ, 640 pp
4.37 Bathe K J, Gracewski S Dec 1981 On non-linear dynamic analysis using substructuring and mode superposition *Comput. & Struct.* **13**: 699–707
4.38 Du Preez, R J 1973 Review of basic theory and methods of analysis of continuous structures *Symp. Bridge Deck Analysis* South African Institution of Civil Engineers, Kyalami Ranch, South Africa, May 1973, paper no 2
4.39 Strang G, Fix G J 1973 *Analysis of the Finite Element Method* Prentice-Hall, Englewood Cliffs, NJ, 320 pp
4.40 Cusens A R, Pama R P 1975 *Bridge Deck Analysis* Wiley, New York, 278 pp
4.41 Cusens A R 1971 Box and cellular girder bridges: a state-of-the-art report *Symposium on Concrete Bridge Design* vol I, American Concrete Institute, SP-26, pp 272–319
4.42 Aneja I, Roll F 1971 Experimental and analytical investigation of a horizontally curved box-beam highway bridge model *Symposium on Concrete Bridge Design* vol I, American Concrete Institute, SP-26, pp 379–410
4.43 Pama R P, Cusens A R 1971 A load distribution method of analysing statically indeterminate concrete bridge decks *Symposium on Concrete Bridge Design* vol I, American Concrete Institute, SP-26, pp 599–633
4.44 Salse, E A 1971 Analysis and design of prestressed concrete circular bow girders for bridge structures *Symposium on Concrete Bridge Design* vol II, American Concrete Institute, SP-26, pp 714–40
4.45 McMillan C M 1973 Analysis of slab bridges by finite elements *Symp. Bridge Deck Analysis* South African Institution of Civil Engineers, Kyalami Ranch, South Africa, May 1973, paper no 4
4.46 Du Preez R J 1973 Analysis of slab bridges by the finite strip method *Symp. Bridge Deck Analysis* South African Institution of Civil Engineers, Kyalami Ranch, South Africa, May 1973, paper no 3
4.47 Houlding S W 1973 Analysis of box section bridges *Symp. Bridge Deck Analysis* South African Institution of Civil Engineers, Kyalami Ranch, South Africa, May 1973, paper no 13
4.48 Du Preez R J 1973 Analysis of box girders by folded plate and strip methods *Symp. Bridge Deck Analysis* South African Institution of Civil Engineers, Kyalami Ranch, South Africa, May 1973, paper no 14
4.49 Freese C E, Tracey D M Jun 1981 Adaptive load incrementation in elastic–plastic finite element analysis *Comput. & Struct.* **13**: 45–54
4.50 Bathe K J, Ram E, Wilson E L 1975 Finite element formulations for large deformation dynamic analysis *Int. J. Numer. Methods Eng.* **9**: 353–86
4.51 Meyer C, Bathe K J 1980 Non-linear analysis of concrete structures in engineering practice *American Society of Civil Engineers Annual Convention and Exposition* Hollywood-by-the-Sea, Florida, Oct 1980
4.52 Bathe K J 1982 *Finite Element Procedures in Engineering Analysis* Prentice-Hall, Englewood Cliffs, NJ, 735 pp
4.53 Ferritto J M 1979 An evaluation of the ADINA concrete model *Proc. ADINA Conf.* Cambridge, MA, Aug 1979, pp 650–80

4.54 Hannus M 1979 Modelling reinforced concrete structures with ADINA *Proc. ADINA Conf.* Cambridge, MA, Aug 1979, pp 681–91

4.55 Collatz L 1966 *The Numerical Treatment of Differential Equations* Springer Verlag, New York, 535 pp

4.56 Zienkiewicz O C 1977 *The Finite Element Methods* 3rd edn, McGraw-Hill, Maidenhead, 787 pp

4.57 Przemieniecki J S 1968 *Theory of Matrix Structural Analysis* McGraw-Hill, New York, 480 pp

4.58 Avramidis I E, Pohlmann G 1980 Berechnung von ebenem Bogensystemen *Bautechnik* **6**: 193–5

4.59 Davenport A G Jun 1967 Gust loading factors *J. Struct. Div. Am. Soc. Civ. Eng.* **93** (ST3): 11–34

4.60 Davenport A G Nov 1962 The response of slender, line-like structures to a gusty wind *Proc. Inst. Civ. Eng.* **23**: 389–408

4.61 Davenport, A G 1966 The treatment of wind loading on tall buldings *Proc. Symp. Tall Bldgs* Dept Civil Engineering, University of Southampton, April 1966, pp 3–44

4.62 Mitschke H 1967 Beitrag zur Schnittkraftumlagerung am abschnittsweise hergestellten Durchlaufträger aus Spannbeton *Bauingenieur* **42**: 98–100

4.63 Trost H, Wolff H-J 1970 Zur wirklichkeitsnahen Ermittlung der Beanspruchungen in abschnittsweise hergestellten Spannbetontragwerken *Bauingenieur* **5**: 155–69

4.64 Trost H, Mainz B, Wolff H-J 1971 Zur Berechnung von Spannbetontragwerken im Gebrauchszustand unter Berücksichtigung des zeitabhängigen Betonverhaltens *Beton- und Stahlbetonbau* **9**: 220–5 and **10**: 241–4

4.65 Schade D 1977 Alterungsbeiwerte für das Kriechen von Beton nach den Spannbetonrichtlinien *Beton- und Stahlbetonbau* **5**

4.66 Neville A M 1970 *Creep of Concrete: Plain, Reinforced and Prestressed* North Holland, Amsterdam, 622 pp

4.67 Gerstle K H, Aschl H, Belloti R, Bertacchi P. Kostovos M D, Ko H-Y, Linse D, Newman J B, Rossi P, Schickert G, Taylor M A, Traina L A, Winkler H, Zimmerman R M Dec 1980 Behaviour of concrete under multiaxial stress states *J. Eng. Mech. Div. Am. Soc. Civ. Eng.* **106** (EM6): 1383–403

4.68 Salse E A B 1971 Analysis and design of prestressed concrete circular bow girders for bridge structures *Symposium on Concrete Bridge Design* vol II, American Concrete Institute, SP-26, pp 714–40

4.69 Causevic M S 1980 Computing intrinsic values of flexural vibrations of cable-stayed bridges *Final Report 11th Congr. Int. Ass. Bridge Struct. Eng.* Vienna, pp 603–8

4.70 Daiguji H, Okui D, Yamada Y 1980 Interactive and automated design of cable-stayed bridge *Final Report 11th Congr. Int. Ass. Bridge Struct. Eng.* Vienna, pp 615–20

4.71 Arkin A, Bunch-Nielsen T, Petersen S E 1980 Computer design of complex building structures, *Final Report 11th Congr. Int. Ass. Bridge Struct. Eng.* Vienna, pp 621–6

4.72 Kmita J, Bien J, Machelski C 1980 Eine effektive Methode zur Spannungsanalyse, *Final Report 11th Congr. Int. Ass. Bridge Struct. Eng.*

Vienna, pp 627–30

4.73 Fanelli M, Giuseppetti G 1980 Mathematical analysis of structures: usefulness and risks, *Final Report 11th Congr. Int. Ass. Bridge Struct. Eng.* Vienna, pp 637–46

4.74 Geradin M, Idelsohn S, Hogge, M Jun 1981 Computational strategies for the solution of large non-linear problems via quasi-Newton methods, *Comput. & Struct.* **13**: 73–82

4.75 Ito Y M, England R H, Nelson R B Jun 1981 Computational methods for soil/structure interaction problems, *Comput. & Struct.* **13**: 157–62

4.76 Cheng F Y Jun 1981 Inelastic analysis of three-dimensional mixed steel and reinforced concrete seismic building systems, *Comput. & Struct.* **13**: 189–96

4.77 Hsu C-T T, Mirza M S, Sea C S Jun 1981 Non-linear analysis of reinforced concrete frames, *Comput. & Struct.* **13**: 223–7

4.78 Noor A K Jun 1981 Survey of computer programs for non-linear structural and solid mechanics problems, *Comput. & Struct.* **13**: 425–65

4.79 Chung H W, Gardner N J 1971 Model analysis of a curved prestressed concrete cellular bridge *Symposium on Concrete Bridge Design* vol I, American Concrete Institute, SP-26, pp 320–56

4.80 Aneja I, Roll F 1971 Experimental and analytical investigation of a horizontally curved box-beam highway bridge model *Symposium on Concrete Bridge Design* vol I, American Concrete Institute, SP-26, pp 379–410

4.81 Scordelis A C, Bouwkamp J G, Wasti S T Oct 1973 Structural response of concrete box girder bridge *J. Struct. Div. Am. Soc. Civ. Eng.* **99** (ST10): 2031–48

4.82 Scordelis A C, Larsen P K Aug 1977 Structural response of curved reinforced concrete box girder bridge *J. Struct. Div. Am. Soc. Civ. Eng.* **103**: (ST8) 1507–24

4.83 Bouwkamp J G, Scordelis A C, Wasti S T 1980 Failure study of a skew box girder bridge model *Final Report 11th Congr. Int. Ass. Bridge Struct. Eng.* Vienna, 855–60

4.84 Shu-t'ien Li (General Chairman of Symposium) *Symposium on Concrete Bridge Design* American Concrete Institute, vols I and II, SP-26, Detroit, 1971

5 Design and Construction Practice for Concrete Bridges

5.1 General

Modern design and construction practice for concrete bridges is the result of the historical process described in Chapter 1, Section 1.2 and it is clear that the developments, particularly in the last three decades, have very greatly increased the understanding of structures and the actions to which they are subjected. This improved knowledge, together with the powerful methods of analysis that are now available, has had a marked influence on modern practice, which is described in general terms in this section. Other references should be consulted for more detailed information about the properties of materials, design and analysis, construction, reinforcing steel and prestressing details.[5.1–5.10] The more realistic definition and control of material properties using statistical concepts has greatly improved the quality of construction and consequently the scope and reliability of bridges. Innovations in methods of construction have in turn influenced the competitiveness of specific configurations. The conceptual designer must consequently take cognisance of all feasible methods of construction which, in the case of long-span bridges, may radically influence the design.

5.2 Selection and Investigation of Bridge Sites

Usually, the approximate site at which a bridge is to be built is determined by considerations related to the overall planning of the highway, railway or pipeline route and the bridge engineer has the responsibility of finding the best location within the area that the flexibility of the route planning allows. There are, however, many examples where a particularly suitable bridge site was the starting point for the location of the highway, railway or pipeline approaches. This usually applies in the case of river or gorge crossings where suitable bridge sites may exist only at a limited number of places. In the evaluation, the bridge cannot be isolated but must be considered as part of the total system being designed.

In recent years, an awareness of aesthetic considerations and the need for the conservation of the environment and objects of historical value have influenced the location and construction of roads and freeways (see Chapter 1, Sections 1.4 and 1.5). The optimum solution in terms of functional considerations, with due regard to environmental impact studies, may indicate sites which are not necessarily the most suitable from the pragmatic engineering point of view. It nevertheless remains the responsibility of the engineer to obtain sufficient information to determine the most suitable position for the crossing within the constraints imposed on him and to conceive the best structure in terms of the site conditions, so as to achieve the requirements described in Chapter 1, Section 1.5.

In order to evaluate a probable bridge site, many aspects have to be considered, of which the most important are summarized below.

Before proceeding with the investigation, it is advisable to obtain all available information on aspects which will affect the eventual design, including design criteria (see Chapter 2). Use should be made of the following site data and important criteria (where applicable):

(a) route location maps;
(b) topographical maps, including any available aerial and terrestrial surveys and river cross-sections;
(c) local geological or soil maps and any relevant geological and geotechnical information available from neighbouring works;
(d) aerial and terrestrial photographs;
(e) weather and flood records or oceanographic and tidal information where applicable;
(f) location of existing bridges over the relevant river and performance data during floods;
(g) information on ice, debris and channel stability;
(h) land values, land use and expropriation plans;
(i) services location plans;
(j) wildlife habitats and vegetation distribution maps.

5.2.1 Site Investigation

The extent and detail of the investigation of a bridge site depends largely on the class of bridge (see Chapter 2, Section 2.2) and the nature of the sites that are available. The investigation may comprise various stages, starting with a study of topographical maps, surveys, photographs and other sources of information and, by a process of elimination, selecting the most promising sites to be investigated in further detail. Such studies will require knowledge and experience in the interpretation of geological features from aerial or terrestrial photographs and the advice of specialists may have to be obtained.

Where a project may be situated in a foreign country, the engineer should familiarize himself with local customs and the facilities available and investigate the laws and regulations pertaining to the procurement, importation and use of plant, materials and labour, as these matters may eventually influence the selection of the most suitable structure for a particular site.

A detailed inspection of each of the selected sites and sufficient information on them are required before a realistic comparison can be made.

5.2.2 Investigation and Visual Inspection of the Site

At this stage, the engineer should have some idea of the various alternative structures that could be considered. Useful information can only be obtained by a visual inspection if the engineer is properly prepared to observe the relevant features at the site.

A detailed checklist could be drawn up and should cover the following items:

(a) Topography: nature of the approaches to the site, the space to be spanned, suitable abutment and foundation positions, accessibility.
(b) Vegetation.
(c) River and flood characteristics: recorded flood levels, flood marks, scour damage, direction of flow, tendency of river to meander, etc.
(d) Oceanographic and tidal information (if relevant).
(e) Surface geology: rock and soil types, discontinuities in rock strata, i.e. global and local stability of the rock and soil masses, faulting, folding, bedding, joints, weathering, water table and drainage.
(f) Any seismic records or information that may be available.
(g) Obstructions such as structures, pipes, cables or overhead wires that may interfere with operations.
(h) Degree of exposure to weather: wind, rain, snow, icing, and the degree of corrosiveness of the atmosphere.
(i) Available services: electricity, communication services, water supply, highways and railways, access to site and suitable locations for offices, construction plant, stores, yards and camps.
(j) Availability of materials of construction.

The investigation and interpretation of the surface geology will require a thorough knowledge of engineering geology and normally requires the services of a specialist.

At completion of this preliminary investigation, the engineer might be in a position to select the most suitable site which should then be investigated in more detail.

The detailed investigation should be carefully planned to avoid duplication of work or inadequate information on any particular aspect. This may comprise a detailed topographical survey, the compiling of data for hydrologic and hydraulic analyses, subsurface foundation investigations and the investigation of material sources for sand, stone and water.

5.2.3 Topographical Survey of Site

For bridge design, accurate contour plans showing all surface detail of the site, the approaches and surrounding areas, are indispensable. The survey of the site should be tied in with the trigonometric survey coordinate system of the responsible authority.

On difficult terrain, specialized survey techniques comprising aerial and/or terrestrial photography backed up with conventional trigonometric survey methods may be required. In the case of bridge structures over water, special survey and depth-sounding methods may have to be employed, depending on the extent of the areas to be surveyed, the depth of water and the tidal and current flow patterns. Over deep water, initial seismic surveying methods, subsequently substantiated by direct depth-sounding measurements, may be useful.

5.2.4 Hydrologic and Hydraulic Analyses

The hydrologic and hydraulic analyses consist of using various approved methods of establishing maximum probable floods at the site within specific return periods in order to size the waterway under the proposed bridge to pass a design flood of a magnitude and frequency consistent with the risk that is acceptable for the type or class of highway or railway. The procedures are usually prescribed by the authorities. Reference should be made to Chapter 2, Section 2.4.5.

5.2.5 Subsurface Soil Investigation

5.2.5.1 On Land

Where alternative sites have to be investigated and compared, the subsurface investigations may initially be of an exploratory nature only in order to establish which site has the most favourable foundation conditions, i.e. depth and consistency of suitable load-bearing strata. On sites where the number of alternatives is limited to one or two structures, the initial investigation may be planned to cover the actual foundation areas. A detailed investigation would usually only be proceeded with once a structure has been chosen and the actual foundation positions established.

During the preliminary investigation, it should be attempted to retrieve continuous core samples of the substrata which will enable the engineer to form a reliable conception of the various soil and rock formations and to identify areas of weakness and potentially unstable zones. Once the general structure of the subsurface formations have been determined, the engineer will be in a better position to establish suitable foundation positions and to proceed with the planning of the bridge.

The detailed subsurface investigation required for a particular site will depend largely on the conditions at site and the type of structure which is being considered. In all cases, the potential foundation problems should, where possible, be identified at the earliest feasible stage and the detailed investigation for the particular structure planned accordingly.

The optimum amount which should be expended on the investigation of a site will depend on the nature and variability of the soil, the effect on the structure of probable differential settlements and the risk and cost implications of possible foundation failure (see Chapter 3, Section 3.2). The detailed foundation investigation may comprise some of the following procedures:

(a) *Shallow investigations* in hand- or mechanically-dug trial pits or trenches; auger drilling by hand. Tests include:
   1. Plate loading or horizontal jacking tests to determine the load-bearing capacity and the *E*-modulus of soil or rock.
   2. Field shear vane — for undrained shear strength of clayey soil.
   3. Density tests in soil — standard density test or dynamic cone methods to determine relative densities.
   4. Recovery of disturbed and undisturbed samples for laboratory analysis (auger drilling to extend depth of sampling).
   5. Full- or reduced-scale load-testing of prototype foundation.

(b) *Deep investigations in soils*, which include:
   1. Percussive wash boring with 'down-the-hole' tests: recovery of disturbed samples, standard penetration tests (SPT) to measure *in situ* soil consistency, dilatometer (pressuremeter) tests, shear vane and screw-plate tests.
   2. Quasi-static cone penetration and sleeve resistance tests; Dutch cone penetrometer or piezometer probe to determine equivalent point and shaft perimeter resistances of piles, structural properties including undrained shear strength of soils and approximate density of soils; drop-weight cone penetrometer (DCP) tests are ideal for preliminary investigations.
   3. Core boring; with tests as under 1 above, but with the recovery of undisturbed samples for laboratory analysis; or by large-diameter flight auger drill (600 mm plus) for down-the-hole inspection, testing or sampling.

(c) *Deep investigations in rock* by rotary core drilling recovering representative rock samples for visual inspection and laboratory analysis; use of the Goodman jack for determining the *in situ* *E*-modulus.

(d) *Laboratory testing*, which includes:
1. Soil classification tests: grading analysis, Atterberg indicator tests; specific gravity determination.
2. Density, compactibility, consolidometer, direct shear or unconfined and triaxial compression tests; laboratory vane tests.
3. Tests on rock samples: point load indicator, unconfined compression and aggregate suitability tests where required.
4. Chemical tests.

5.2.5.2 Over Water

Subsurface (geophysical) investigations conducted over water may comprise seismic survey methods to establish the overall geology of the site below the waterline. Seismic surveys should be substantiated by core drilling conducted from suitable floating drilling platforms.

It is advisable to use specialists in the field of geology and geotechnical engineering for advice and guidance on all aspects of site investigation where the engineer lacks sufficient competence.

5.3 Methods of Construction and Erection of Bridge Superstructures

As indicated in Chapter 2, Section 2.5, the available construction techniques may have a significant influence on the selection of the most suitable class of bridge for a particular site. It is therefore important that the designer should at an early stage be aware of the various construction techniques that are economically viable, as well as their advantages and disadvantages.

There are three main methods of construction for concrete:

(a) *in situ* casting in formwork in position on the works;
(b) precasting off the works and subsequent transportation and erection;
(c) composite, being a combination of parts as in (a) and (b);

and four main forms of erection:

(a) on centering, i.e. stationary falsework supported directly at ground level or in the form of fixed girders or arches, or travelling falsework supported on the substructure or, when necessary, also on intermediate towers;
(b) cantilevering from previous sections or substructures with or without suspended cable support;

(c) horizontal incremental jacking;
(d) vertical hoisting, lifting or jacking.

5.3.1 Stationary Falsework

Probably the oldest form of construction of concrete structures is by the *in situ* placing of concrete on temporary stationary falsework. Despite the fact that labour and economic conditions vary from country to country, its use is probably still the most widespread means of construction for short- to medium-length bridges (up to about 300 m, 1000 ft) of moderate height (about 10 m, 33 ft). While closely-spaced timber centering was used in the past (some of the timber falsework for arches — now obsolete — were in fact masterpieces of engineering in their own right),[5.11] most stationary falsework today consists of standardized or proprietary steel sections used in towers, frames, girders, etc. Catalogues of various systems showing sizes of struts, frames, girders, couplings and accessories and safe load tables, are readily available in most countries. Points requiring close attention are adequate founding of towers and frames on prepared beds or concrete footings where necessary, and adequate bracing against lateral loads and buckling. The probable settlement of formwork during the placement of concrete should be allowed for and the necessary precamber built in to allow for long-term deformations. Special precautions such as differential jacking in gradual stages may be necessary to avoid cracking of the semi-hardened concrete. Mechanical jacks such as screw jacks, wedge-type jacks or sandpots are suitable. Simple hydraulic jacks that cannot be locked are generally not suitable for supporting long-term loading, because of possible leakage.

Some advantages of a stationary falsework system are the following.

1. It can be erected and dismantled by semi-skilled labour under supervision.
2. It is readily adaptable to intricate and variable geometry of the superstructure.
3. Sections are standardized and suitable for multiple re-use.
4. Sections are of limited size and readily transportable.

Some disadvantages of the system are as follows.

1. The system is cumbersome and slow; therefore, to achieve a reasonable rate of construction, staging may have to be erected for one or two spans ahead of the span under construction.
2. It requires relatively firm founding conditions which may render it unsuitable for certain sites. It is not very suitable for river crossings where the additional risk of floods may exist.

3. In built-up areas the need to maintain traffic may be a problem unless a special underpass section is provided. This is more costly and includes the risk of vehicular collision.

Examples of stationary formwork are shown in Figs 5.1 and 5.2. This form of staging is most commonly used for *in situ* concreting, but can also be used for precast segments.[5.12]

5.3.2 Travelling Falsework

Some of the disadvantages of stationary falsework can be overcome by the use of travelling formwork. The simplest form is longitudinally travelling formwork. For limited lengths of construction, the staging consists of scaffold towers with beams or girders supporting the formwork. On stripping, the formwork is lowered, the soffit shutters in the line of the piers swing aside and the whole assembly is pulled forward by winching to the next span. For this method, firm ground conditions and a reasonably constant height above ground level are essential. If the spans are simply supported, the staging extends for one span. If a continuous superstructure is constructed in stages, the construction joint is usually provided at approximately the point of contraflexure. When the formwork–falsework assembly is transported to the next span, it is rigidly connected to the cantilever of the previous span (Fig. 5.3).

Figure 5.1 Mainbrücke Bettingen, Frankfurt (courtesy of Polensky and Zöllner)

Figure 5.2 Bridge over the Vaal River, South Africa

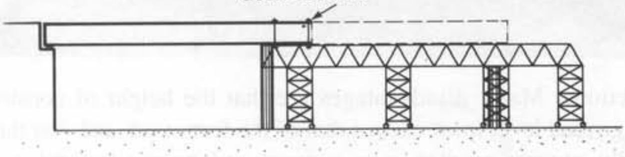


Figure 5.3 Travelling formwork on rollers

If the ground conditions are poor, the ground level is variable or the bridge is high above the ground, movable casting girders which are supported off the permanent sub- or super-structure can be a viable alternative solution. Casting girders can be classified into three types:

(a) the girder is above the final superstructure;
(b) the girder is below the final superstructure;
(c) the girder extends above and below the final superstructure.

The principal advantage of the 'girder above' solution is that it can be used for spans where the height of construction above ground level may be small. It is also suitable for the launching of precast elements. A major disadvantage is that the formwork normally requires temporary suspension rods passing through the superstructure.

The 'girder below' solution has the advantages that no suspension rods penetrate the superstructure and the deck area can be kept free of

Figure 5.4
Draaiberg Bridge near Cape Town, South Africa (courtesy of R. Meuwese)

obstructions. Major disadvantages are that the height of construction above ground level must exceed that of the formwork and that the piers normally require adaptation to support and permit launching of the girder.[5.13] If the spans are large, use can be made of auxiliary temporary towers to limit the girder height, or the girder can be stiffened by means of prestraining with a king post and adjustable inclined ties, as in Fig. 5.4.

The girder shown in Fig. 5.4 was used to construct a continuous bridge in 40 m (130 ft) stages. The main casting girder consisted of four steel trusses. The central two trusses were provided with lighter structural steel launching noses and tail ends. The total length of girder from nose to toe was 92.73 m (304 ft) and its mass 190 tonnes (210 US tons). In a typical span, the side girders were supported on structural steel brackets stressed with prestressing rods to the outside of the pier stub columns. The construction technique was as follows.

1. After the curing of the deck, the two main beams were post-tensioned longitudinally.
2. The girders were lowered by wedge jacks on to rollers at the piers. This also released the shutters from the deck soffit.

3. The king post and ties were uncoupled and fixed to the girder underside.
4. The beam soffit shutters were swung aside to clear the pier stub columns and the girder launched forward by means of hydraulic jacks. Cross-bracing had to be removed and replaced as it passed the piers.
5. The girder was set up on the wedge jacks, king post and ties reinstated, shutters realigned and precamber set ready for the construction of the next stage.

Another variation of the 'girder under' operates on the slide-rule principle (Fig. 5.5). The system consists of three girders — a central launching and casting girder and two outer casting girders. The central girder is supported on the pier between stub columns and extends over three piers. The outer girders extend for the span of the stage under construction. At the rear end they are supported by means of a cross-girder on the completed section of the bridge. At the front end they are supported by means of a cross-girder on the central launching girder and on temporary brackets stressed to the front pier. During launching the outer girders are supported on the central girder at the front and the completed deck at the rear. The temporary pier brackets are transported forward with the outer girders during launching.

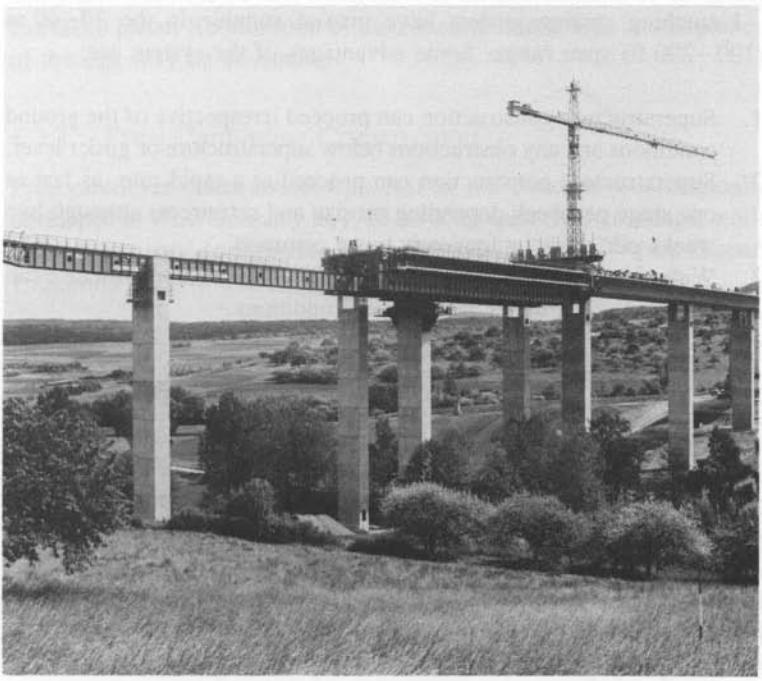


Figure 5.5 Vinxtbachtal-brücke, Cologne (courtesy of Polensky and Zöllner)

Figure 5.6 MRT viaducts, Singapore (courtesy of Dr P Marti, VSL International)

There are many other variations which are beyond the scope of this handbook; for further details see Refs 5.13–5.16 (see also Fig. 5.6).

Launching casting girders have proved suitable in the 30–60 m (100–200 ft) span range. Some advantages of the system are:

1. Superstructure construction can proceed irrespective of the ground conditions and any obstructions below superstructure or girder level.
2. Superstructure construction can proceed at a rapid rate, as fast as one stage per week depending on size and resources, although two weeks per stage or longer is more common.
3. With suitable design of cover to the girder, the superstructure can be constructed under factory site conditions.

Some disadvantages of the system are:

1. Launching casting girders require a fairly high capital investment, the cost of which must often be amortized on one or only a limited number of structures. Typical masses of steel per ton of concrete are 300–600 kg (660–1320 lb).[5.13] The material content of the girder shown in Fig. 5.4, relative to the structure, was 250 kg steel/1000 kg concrete (552 lb/2208 lb).
2. Launching girders are not readily adaptable to changes in superstructure cross-section and sharp radii of curvature.

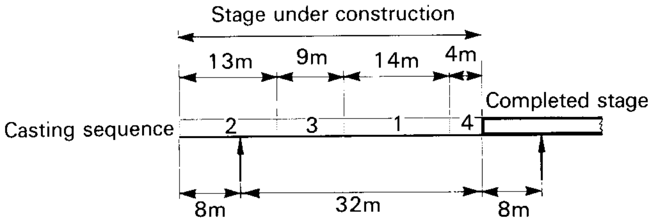


Figure 5.7 Casting sequence employed to avoid excessive imposed strains in hardening concrete

3. Launching casting girders, for reasons of economy, are usually highly stressed and flexible relative to the bridge superstructure. Therefore, to ensure a satisfactory final profile of the superstructure, the deflection of the girder during casting should be accurately assessed so that the correct precamber can be applied. Temperature effects, if significant, should be taken into account. The casting procedure and sequence should be chosen in such a manner that the hardening concrete does not suffer damage due to excessive imposed strains. Concrete set retarders might be advisable. Figure 5.7 shows the casting sequence employed to overcome this problem on a particular structure where the achievable casting rate was slow (about 80 m^3/day, 105 yd^3/day).

Note: It is important that the section of new concrete adjacent to the completed structure be cast when most of the deformation of the girder has taken place. Re-vibration of the concrete in this area at completion of the cast may be advisable.

5.3.3 Free Cantilever Construction

Free cantilever construction with cast *in situ* concrete was originally developed in West Germany in 1950 for long-span river crossings, while precast segmental construction was pioneered in France[5.17] (see Chapter 1, Section 1.2).

5.3.4 *In situ* Free Cantilever Construction

In its basic form, *in situ* free cantilever construction proceeds as shown in Fig. 5.8. After construction of the deck section over the pier, cantilevering sections are cast progressively in segments, usually symmetrically about the pier to control the unbalanced moment acting on the pier. The length of segment cast depends on economic factors determined by the mass of concrete cast in one cycle and the cost of the carriage. Segments of 3–5 m are common. After the hardening of the concrete, the segment is post-tensioned to the completed portion to

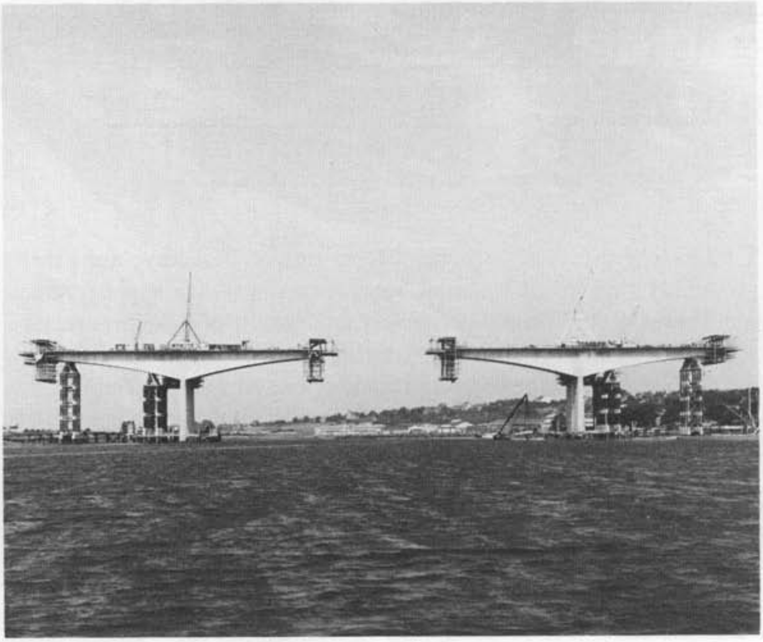


Figure 5.8 Medway Bridge, England (courtesy of Freeman, Fox and Partners)

carry the cantilever moment. As construction proceeds, the continuity cables are inserted progressively and stressed across various segments. Before the last concrete that closes the central gap is cast, the two cantilevers are locked together by various methods to avoid damage to the hardening concrete due to movements induced by temperature changes or external actions. As the concrete in the gap hardens, the sagging moment cables are inserted and stressed in stages (Fig. 5.9). In this way, construction proceeds from span to span and the structure is made continuous. A major advantage of *in situ* cantilever construction is the continuity of reinforcement across the construction joints. The method of construction has been successfully used for spans of up to 240 m (787 ft) — for example, the Hamana–Chasi Bridge in Japan.

It is important for stability that the deck be fixed to the piers during construction. If the pier height is suitable, it can be made monolithic with the superstructure. Alternatively, the deck can be stabilized with temporary supports or the bearings at the top or bottom of the piers can be fixed during construction. The latter technique was adopted for the Mosel Bridge at Schweich, for instance.[5.18]

A variation of the free cantilever construction method is to use launching girders which can serve as supports during construction while also being used for the transport of men, materials and the carriage. The method was first employed during the construction of the Siegtal Bridge.[5.19]

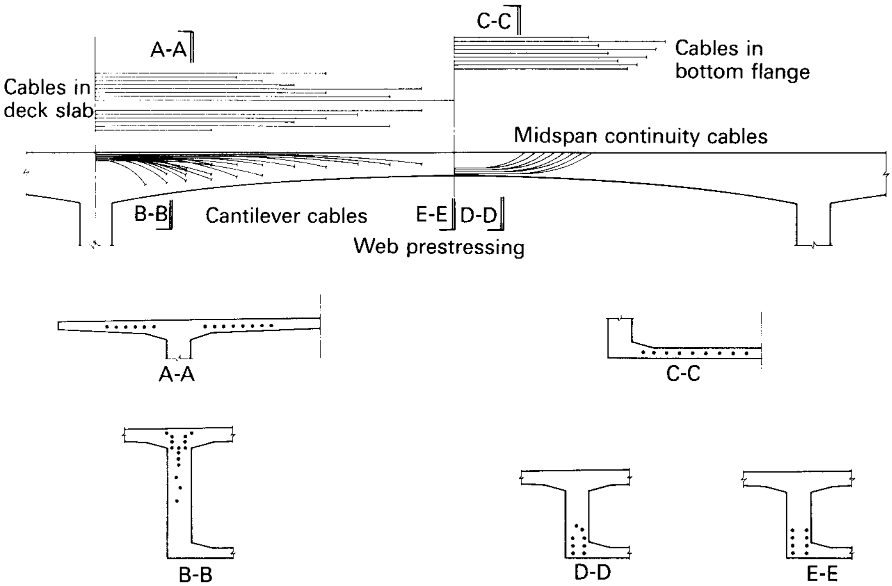


Figure 5.9 Prestressing details

To limit the cantilever moments, the superstructure can be constructed in stages, e.g. the box section is constructed by the free cantilever method and the deck cantilever slabs are made in subsequent operations (Eschachtal and Kochertal Bridges.)[5.20] In certain instances, the use of temporary auxiliary towers offers a practical solution (Medway Bridge in England, Fig. 5.8). Auxiliary pendulum towers and temporary tie-back cables have been used for the cantilever construction of continuous bridges. The method is eminently suited for the construction of arch bridges and strut-frames over deep gorges (see Section 5.6 and Figs 5.10 and 5.11).

5.3.5 Precast Segmental Free Cantilever Construction

The segmental construction proceeds similarly to the *in situ* free cantilever construction with the following major differences. The superstructure is divided into segments which are precast in suitable sizes as dictated by transport and handling facilities. These segments are then transported to site and erected, commencing with the section over the pier and proceeding by cantilever construction from both sides of the piers. An important feature is the treatment of the joints between the elements. There are two basic methods:

1. The joint is cast *in situ*, of adequate size to permit sufficient overlap for the effective continuity of longitudinal reinforcement. This

Figure 5.10
Bloukrans Bridge, South Africa

Figure 5.11
Gouritz River Bridge, South Africa

method was employed for the construction of the Oosterschelde Bridge in the Netherlands.[5.11]

2. The most common form of joint treatment is by 'glueing' the segments together and forming a very thin joint. No ordinary unstressed reinforcement can accordingly cross the joint. To achieve the close tolerances required, match-casting is done by casting the element under construction against the preceding one. The elements are then erected in sequence. Prior to fitting, the contact areas are coated with a bonding agent, generally an epoxy, sometimes a cementitious product. This bonding agent is not usually taken into account in the structural design as contributing to the ultimate strength of the section. However, in the case of a segmental bridge constructed on staging over the Europa Canal near Nuremberg, the bonding agent was considered as contributing and tests were carried out to assess its effectiveness.[5.21] The functions of the bonding agent are: (a) to lubricate the matching segment faces so as to facilitate alignment; (b) to take up any minor misalignment which, despite match-casting, occurs mainly because of differential shrinkage, creep and temperature; and (c) to ensure that the joints are solid and watertight.

The faces of the joints were previously provided with shear keys, which served to align the segments and transfer shear stresses and diagonal compressive stresses. Shear keys can give rise to excessive stress concentrations. The modern method of profiling the joints is by means of sawtoothing and providing shallow keys for alignment purposes in the webs[5.17,5.22] (see details of the Sallingsund Bridge).[5.23] The fact that the unstressed reinforcement is discontinuous over the construction joints requires full prestressing for moments due to loads, temperature and secondary moments. For the former reason, the theoretical ultimate moment of resistance is reduced.[5.24] On the other hand, the construction method is speedy and this serves to offset the additional cost of prestressing. In both the *in situ* and precast cantilever construction methods, the deflections due to loading, creep, differential shrinkage and temperature have to be carefully assessed so that both the structural and geometric requirements are complied with.[5.25]

5.3.6 Incremental Launching

The incremental launching method, which was developed by Leonhardt and Baur,[5.26] consists of constructing the bridge superstructure in stages by casting successive segments at one end of the bridge. The segments are usually 15–30 m (50–100 ft) long. After the segment under construction has gained sufficient strength it is prestressed for the launching stage and the completed sections of the bridge launched one stage forward and the procedure repeated. During launching, the section undergoes complete stress reversals as it progresses from a cantilever to the first support and thereafter over the following spans to its final position. For launching from one end, the bridge profile should be constant, preferably straight, but horizontal and vertical curvature of constant radius can be accommodated within limits. The deck slab should have a constant superelevation without any transition areas. Minor variation in deck width, especially near the bridge ends, can be accommodated.

Because of the stress reversals during launching, a box section with nearly symmetrical section properties is the most efficient. Other sections, such as a double tee,[5.27] have been used. To cope with the launching stresses, the section is usually post-tensioned axially with cables in both the top and bottom flanges. To limit the cantilever stresses, the front end is usually fitted with a lightweight structural steel launching nose. If the spans are excessively long or variable, temporary auxiliary towers may be necessary. The towers should be designed so that their stiffness is compatible with the launching requirements. Use can be made of jacks to control the support reactions and thus the stresses in the superstructure. The additional permanent state cables are usually threaded in or fixed subsequently and post-tensioned after the bridge has reached its final position. Some of the cables required during launching could be recovered, but it is rarely economic to do so. As the final set of cables is therefore, in effect, required to cater for the range of stresses in the completed structure, this method requires more prestressing. Under favourable conditions, this is more than compensated for by saving in centering and formwork and by the speed of erection. The capital investment in equipment is not very large, with the result that the method has proved to be very competitive.

Figure 5.12 Incremental launching

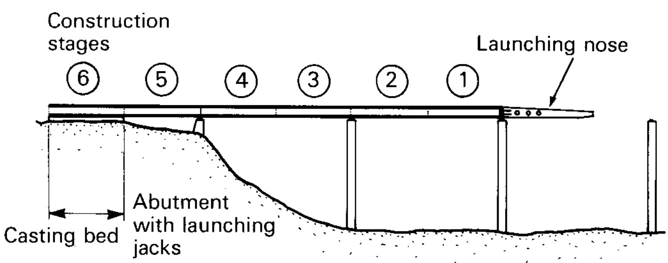
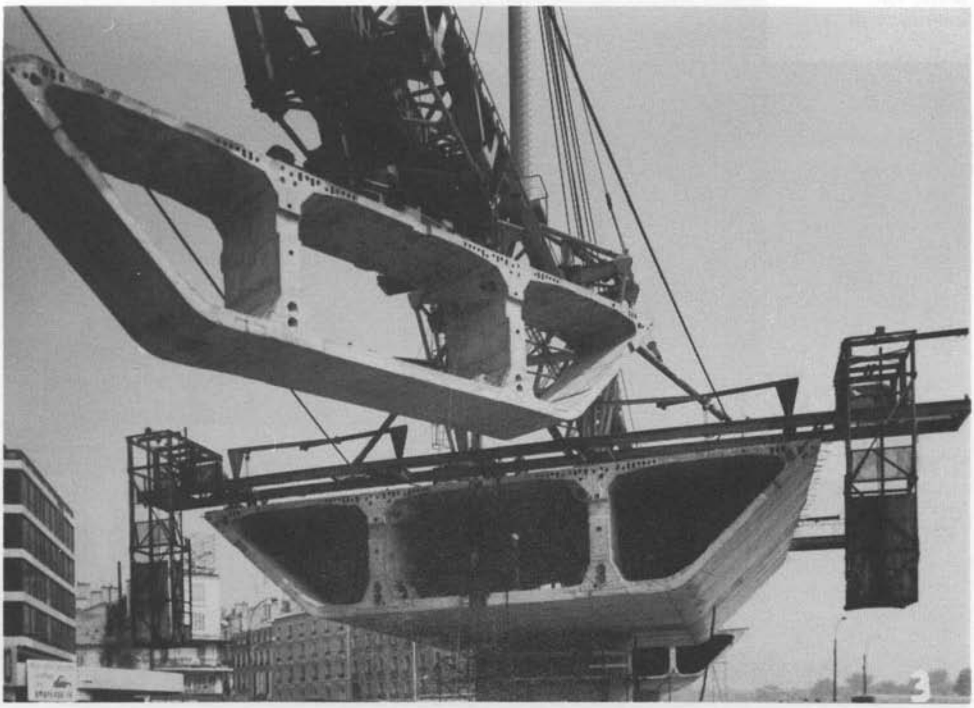


Figure 5.12 shows a diagrammatic elevation of a bridge under construction by the incremental launching system. The first section is poured on the casting bed behind the abutment on the fill. Provision for accurately controlling the soffit levels is essential. To ensure a smooth launching operation, the soffit and edge of the superstructure must be

to the correct profile without kinks and discontinuities. The launching nose is stressed to the first section after it has been axially stressed in preparation for launching. One method of launching is by means of hydraulic jacks fixed to the abutment, which pull the bridge segments forward by means of stressing rods which are temporarily anchored to the section just completed. An alternative method[5.26] involves using a proprietary system of jacks in which the superstructure at the abutment is lifted and then jacked forward, lowered, the launching jacks retracted and the operation repeated to move the superstructure forward in small increments. The piers, abutment and supports of the section under construction are provided with temporary sliding bearings made up of steel or concrete with a stainless steel surface and side guiding plates to keep the bridge on line. Steel reinforced Neoprene bearing plates, usually of 13 mm ($\frac{1}{2}$ in) nominal thickness and coated with Teflon on one side to facilitate sliding over the stainless steel, are placed on each bearing. As the plates move forward, new ones are inserted immediately behind. When the front ones come free, they are re-used at the back. The coefficient of friction is approximately 2 per cent, but the launching equipment and structure should be designed for at least 4 per cent friction.[5.26] If the bridge is on a downward slope, the launch is usually in the downward direction. Slopes greater than about 2 per cent require

Figure 5.13 Precast segment being hoisted (courtesy of Dr P Marti, VSL International)



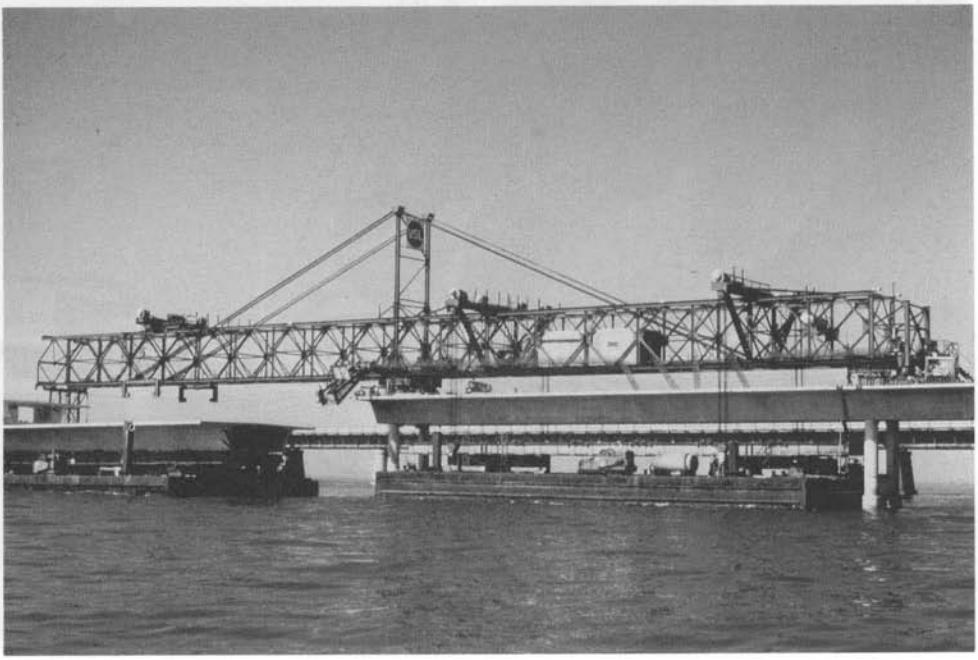
braking devices. The piers must be designed so that the temporary bearings can be replaced by permanent ones. Modern practice is to adapt the permanent bearings to serve as temporary sliding bearings.[5.27] The usual cycle per stage is one week.

5.3.7 Hoistings, Lifting or Jacking

The hoisting, lifting or vertical jacking of precast beam units is commonly used (see Fig. 5.13). Depending on the equipment available and economic considerations, use may be made of cranes or vertical jacks in conjunction with towers, girders or slings. Crossbeams and decks may then be cast *in situ* or precast and similarly placed in position. Entire spans have been precast and jacked into position in this way, e.g. the Seven Mile Bridge in the Florida Keys (Figs 5.14 and 5.15).

A variation of the jacking technique is to construct the bridge, such as a subway crossing, alongside the embankment under which the subway is to pass and then progressively to jack the completed structure into the embankment excavation. The end of the structure which is jacked into the embankment is normally provided with cutting edges not unlike tunnelling.[5.28]

Figure 5.14 Seven Mile Bridge, Florida Keys, USA (courtesy of Dr P Marti, VSL International)



5.4 Bridge Foundations and Substructures

The functions of the bridge substructure are to support the superstructure at the required levels, to carry the dead and live loads safely from the superstructure through the foundations to the subsoil and to provide stability against all actions on the structure. Substructures are designed either to restrain or to accommodate horizontal and rotational movements of the superstructure. Where soil investigations predict settlements or heave of an appreciable order, the effects thereof are to be taken account of in the design of the total structure. In some bridges, monolithic construction of the superstructure and substructures is feasible.

Figure 5.15 Seven Mile Bridge in the Florida Keys, June 1981 (courtesy of Figg and Muller Engineers Inc)

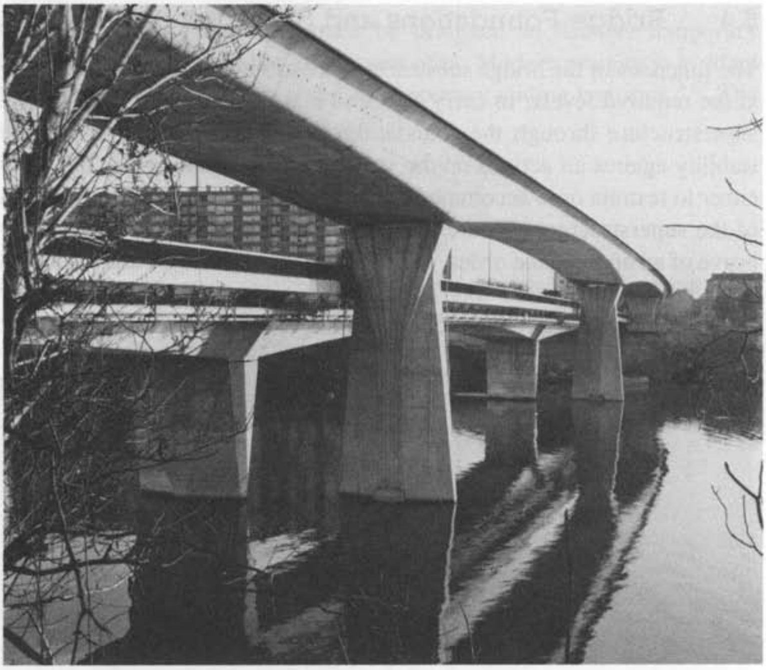


Figure 5.16 Clichy Bridge, France (courtesy of Freyssinet International)

Substructures and foundations are important and substantial parts of bridges, both functionally and costwise. The aesthetic treatment of substructures in relation to the whole structure is very important (Fig. 5.16).

5.4.1 Abutments

Abutments or bank seats support the ends of bridges. The various types of abutment commonly used are listed and illustrated in Chapter 1, Section 1.3. They are usually constructed by conventional means of *in situ* concrete. The superstructure may be integral with the abutment, as in portal frames or diaphragm walls or it may be pinned or supported on bearings with various degrees of freedom. In the case of small structures, the superstructure can be fixed to the abutments by means of dowels which permit small relative rotations, but restrain large horizontal displacements (Fig. 5.17). An abutment must be adequately designed to resist all combinations of forces acting on it, including the earth pressure of the fill it retains. Simplified procedures are prescribed in codes by most authorities. These may under certain circumstances underestimate the resultant forces due to earth pressures, which may exceed that of the 'earth pressure at rest'. This applies particularly to backfill between counterforts. The fill is compacted in layers, each layer

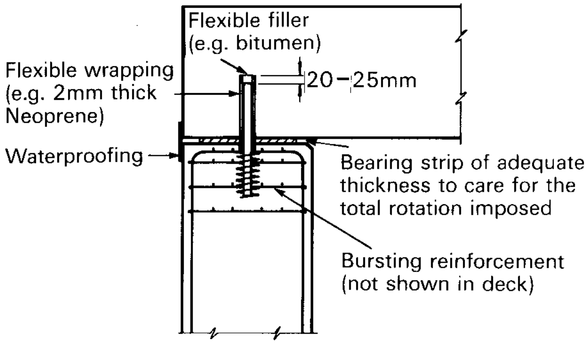


Figure 5.17 Dowel detail for small structure

imposing increased pressures on the wall which will exceed active pressures if the walls do not yield. The fill is furthermore compacted by traffic and in the case where horizontal movement between super- and sub-structure is restrained, the expansion of the superstructure may induce further increases in horizontal earth pressure. Care is therefore required in the design of wall elements to ensure that the basic design assumptions are realized or otherwise to allow for probable increased forces.

When the horizontal movement is too large to permit the superstructure to be pinned to both abutments, the deck is supported on bearings that allow freedom of movement and rotation. In the case of medium-length bridges, it is preferable to fix the deck at one abutment and to provide for freedom of movement at the other. This saves one expansion joint (see Section 5.8). Beyond a certain length of deck, longitudinal movement may have to be allowed for at both abutments, and fixity provided at one or more of the intermediate supports.

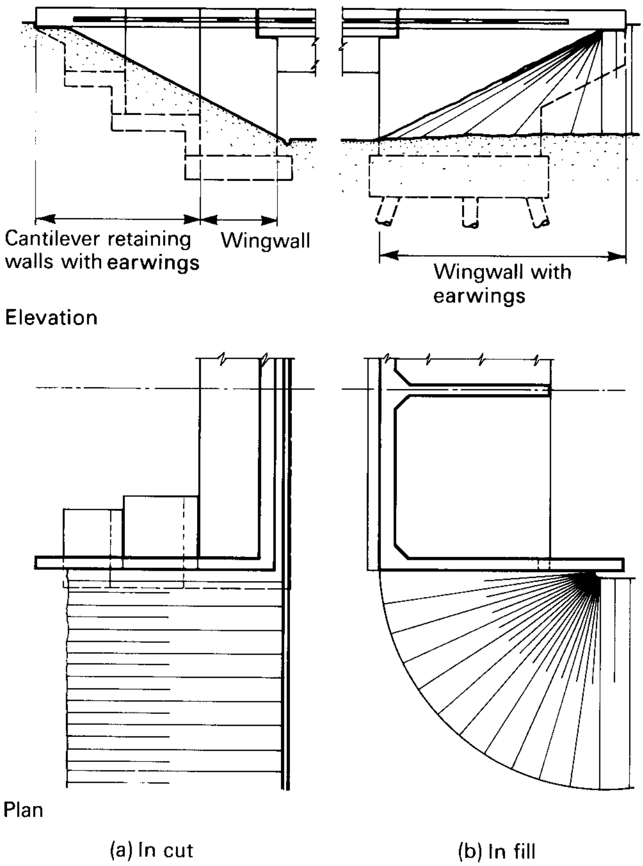
No decks should depend entirely on friction for horizontal stability. Unless sufficient horizontal support is provided, it is imperative in areas where floods or earthquakes may occur, or in the case of sloping decks, to allow for 'stops' beyond the limits of movements required for temperature and other effects.

5.4.2 Wingwalls

The wingwalls of the abutments are extensions that serve to contain the fill from spilling forward or sideways at the ends of the abutment wall. In closed abutments, they can be either parallel to the roadway or continuously in line with the abutment wall, or skewed between these planes (Fig. 5.18). In spill-through abutments, wingwalls (if used) usually cantilever from the breast beam and limit the amount of spill that takes place.

The walls of closed abutments act as deep beams vertically and

Figure 5.18
Closed abutments: (a) in cut; (b) in fill



horizontally, and as slabs in resisting forces normal to their planes. Wingwalls primarily act as cantilever slabs supported at one or two edges, depending on their size and type. Design tables for assessing the moments are available in various publications, e.g. Refs 5.29 and 5.30. For reasons already stated, the horizontal earth pressures should not be underestimated. Leonhardt[5.24] recommends between 1.5 and 3 times active pressure. If the wingwall moments or deflections become excessive, splitting the wingwalls into separate walls might be advisable (Fig. 5.18a).

5.4.3 Approach Slabs

The approach slabs for bridges are constructed under the roadway of highway bridges to avoid a sudden step between the end of the bridge and the fill due to settlement of the latter. The approach slabs are supported on the abutment at one end and rest on the fill towards the other end. The length of approach slab depends on the expected settlement

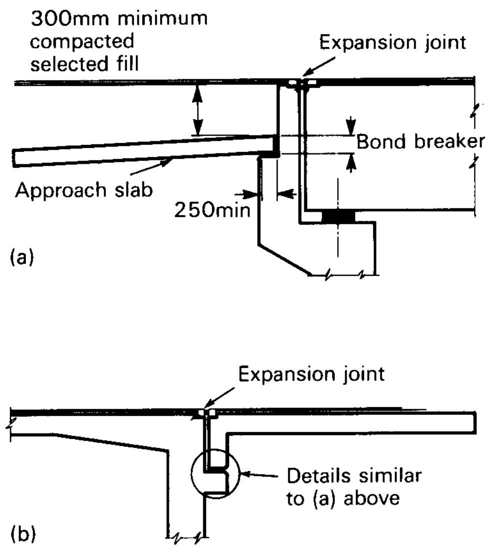


Figure 5.19 Two types of approach slab: (a) buried infill; (b) onfill

of the fill and the acceptable change of gradient in the road profile. Figure 5.19 shows two types of approach slab.

5.4.4 Drainage

The fill behind closed abutments and walls should be drained to prevent build-up of water pressure on the walls and pumping and failure of the fill. In addition, the surface water from the deck might have to be led away via the piers or abutments. Figure 5.20 shows a detail for draining the fill. Surface water from the deck, namely stormwater run-off plus seepage water through expansion joints, should be led via drainage pipes to discharge outlets. Externally mounted pipes are usually aesthetically unacceptable. Pipes cast into the concrete should be sufficiently rigid

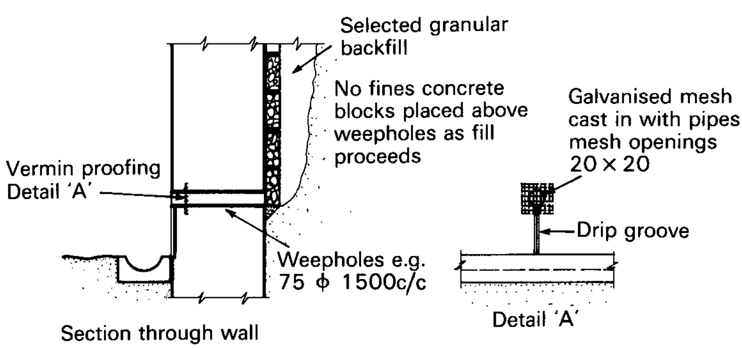


Figure 5.20 Abutment detail for draining fill

and strong to prevent damage and blocking during casting. They should be detailed to allow cleaning by rodding, i.e. have no sharp bends and be sufficiently large to prevent clogging up. A diameter of 100 mm (4 in) is probably a minimum practical size. In areas subject to freezing temperatures, the pipes might have to be sleeved to prevent the concrete from bursting because of the freezing of blocked water.

5.4.5 Piers

Piers form the intermediate supports for bridge superstructures consisting of more than one span. They are usually constructed in *in situ* concrete, either by conventional means or, for tall piers, by continuous or intermittent sliding of the formwork. Precasting is rarely used. The superstructure may be monolithic with the pier or may be supported on bearings with varying degrees of freedom. The same variations may apply at the connection with the foundation, and altogether there are many possible combinations to suit the requirements.

The number and spacing of piers are usually determined by the economics of the total structure. The choice of pier type and shape is influenced by several factors. Aesthetic considerations are very important but other considerations may dictate the basic shape. This applies particularly if the bridge crosses flowing water. The shape and orientation of the piers affect the flow characteristics through the river crossings.[5.31] Wall-type piers with shaped noses are preferred for their hydraulic efficiency. In rivers where the likelihood of floating debris exists, closely spaced piers are normally not desirable. Figure 5.22 shows typical river piers A to D offering decreasing resistance to flow. It has, however, been observed that sharp edges on piers, as in shape D, are

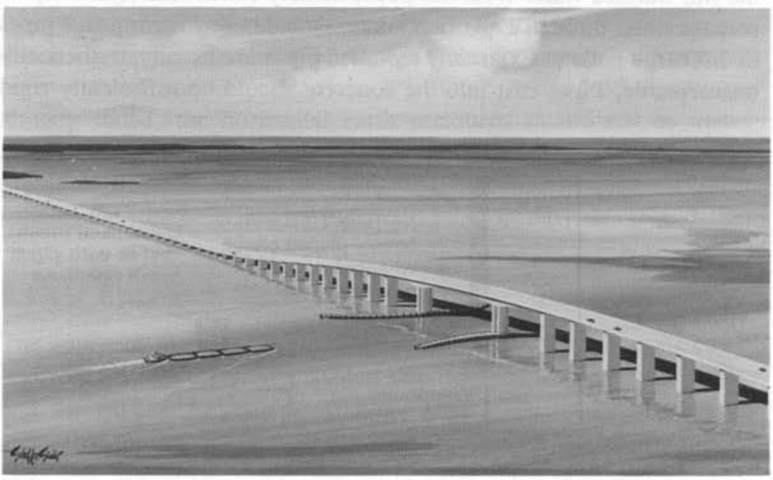


Figure 5.21 Dauphin Island Bridge (courtesy of Figg and Muller Engineers Inc)

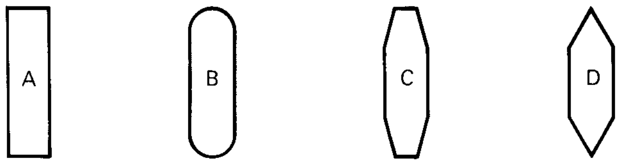


Figure 5.22
Typical sections of piers in rivers

more prone to trapping debris such as branches than well-rounded ends, as in type B. This increases the resistance to flow. Wall-type piers are usually required to be not too slender for aesthetic reasons. Type C, because of shadow effects, can from an angle appear to be as thin as the nose ends. Piers near navigable channels are normally massive to resist impact from shipping. Additional barrier islands are essential in the case of large bridges spanning major shipping routes (Fig. 5.21).

Wall-type piers are not very suitable for road crossings as they give rise to a visual tunnel effect. Isolated columns are preferable. They should be limited in number in the transverse direction for improved appearance and sight distance. However, impact on slender columns can cause catastrophic damage. Impact effects on columns are extremely complex and research on the subject is an ongoing process.[5.32] Reinforcing the compression zone against destruction has proved successful. Large-diameter columns, on the other hand, look very clumsy. Figure 5.23 shows various pier shapes and cross-sections (see also Figs 1.18 and 5.16).

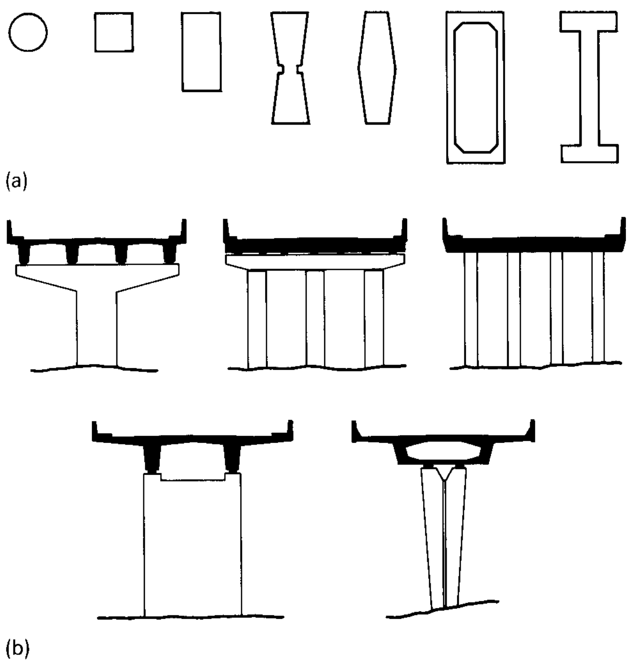


Figure 5.23
Typical (a) pier, and (b) bridge, cross-sections

5.4.6 Foundations

An in-depth discussion of foundations is beyond the scope of this handbook and the reader is referred to specialized literature covering the geotechnical aspects (soil and rock mechanics). Foundations can be classified into two types: 'shallow' (up to say 6 m below ground level), and deep. Examples of the former are spread footings, strip footings and raft foundations. They are founded directly on the substrata. Loose subsoil can be consolidated artificially by mechanical means such as vibroflotation and fissured rock by grouting, rock anchors, etc.

Deep foundations may be in the form of piles or caissons. They transmit the forces down to load-bearing strata (end-bearing) or utilize skin friction, or a combination of both. In river crossings, they must be adequately founded well below the probable scour depth; see Chapter 28 by Mitchell in Ref. 5.1. Small-diameter piles in soil that may be subject to scour should be checked for stability and buckling. See also Chapter 31 by Thomlinson in Ref. 5.1 on piles, and Chapter 3, Section 3.2.6 on the reliability of pile groups.

It is advisable to employ specialists in the field of geology and geotechnical engineering on all sites that may present problems.

5.5 Slab, Beam and Box-Girder Superstructures in Bridges

5.5.1 General

The choice of superstructure is affected by several considerations. Listed here are some of the more important ones: (a) topography of site; (b) geometry of superstructure, i.e. straight, curved, skew, varying or constant width, etc.; (c) required permanent open space under deck; (d) length of spans; (e) founding conditions; (f) type of live load and its magnitude, e.g. road, railway, seismic; (g) available depth of construction; (h) available construction methods; (i) restrictions during construction, such as maintenance of traffic.

Increasing the length of span increases the cost of the superstructure but for multispan structures there is a reduction in the number of piers and foundations. Generally, for a given set of circumstances, it is found that there is a range of economic span lengths. Figure 5.24 shows the relative cost of a road-over-river bridge with deep-piled foundations in relation to span length. The curve is flat over a reasonable span range. The designer can thus usually optimize both cost and aesthetic requirements. Other considerations are the degree of continuity of spans, i.e. simply supported, partially continuous, or fully continuous. Helminger[5.33] has tabulated the absolute and relative material content for 188 bridge superstructures built in Bavaria in the early to mid-1970s.

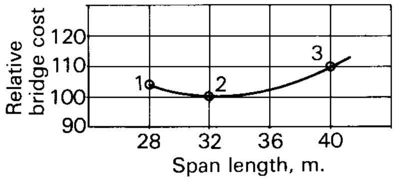


Figure 5.24 Relative cost of road-over-river bridge with deep piled foundations: (1) four-beam, total length 588 m, $l/h = 14$; (2) eight-beam, total length 576 m, $l/h = 20$; (3) two-beam, total length 584 m, $l/h = 15$; where l/h is the span-to-depth ratio

The various deck configurations that are commonly used are listed in Chapter 1, Section 1.3. The design and detailing of slabs and beams in general are dealt with in Chapters 12, 13, 16, 19 and 20 of Ref. 5.1 and Ref. 5.34.

5.5.2 Slabs

5.5.2.1 Solid Flat Slabs

The solid concrete flat slab is commonly used for short, simply supported spans and up to medium lengths for continuous spans. It is especially suitable for skew crossings or shapes that are variable in plan. Spans up to approximately 15–17 m (49–56 ft) are commonly constructed in reinforced concrete. Larger spans are usually prestressed. Leonhardt[5.24] considers the practical limit for solid slabs to be approximately 20 m (66 ft) for simply supported spans, 30 m (98 ft) for continuous spans and 36 m (118 ft) for continuous spans with haunches. Larger spans have, however, been built.[5.33]

Solid flat slabs can be regarded as isotropic, i.e. having the same physical properties, such as stiffness, in all directions. The effect on the stiffness of varying reinforcement in different directions is usually negligible under service loads. A slab is considered to be anisotropic when these properties differ substantially in various directions. If the properties differ in two specific orthogonal directions, the slab is orthotropic.

A slab loaded perpendicular to its plane can be subject to bending and twisting moments in all directions. These moments and twisting moments have to be resolved into principal moments and their directions determined for evaluation of reinforcement. To do this, the moments about two perpendicular axes and the twisting moment at any specific point is required, i.e. M_x, M_y, M_{xy}. In rectangular one-way spanning slabs, the principal moments due to uniformly distributed loads are approximately parallel and perpendicular to the edges of the slab at midspan and over supports in continuous slabs. In skew slabs and slabs of odd shape, this is not necessarily the case. Under non-uniform superimposed loads, the direction depends on the distribution and location of the loads.

Slabs are usually analysed by the following methods (see Chapter 4):

1. Influence surfaces or tables such as those published in Refs 5.29, 5.35–5.44.
2. Grid analysis. The layout of the grid and correct simulation of member properties is important.[5.3]
3. Finite strip method in certain cases only.
4. Finite element and finite difference methods.
5. Model analysis (see Chapter 23 by White in Ref. 5.1).

Methods 3, 4 and 5 can be relatively expensive and should only be used when the complexity or importance of the structure warrants their use.

In skew slabs, the end support reactions differ with the degree of skewness and stiffness and spacing of bearings. Stiff linear support conditions give rise to high support reactions at the obtuse corner. The reactions at the acute corners can be negative. Skew slabs often require shear[5.24] reinforcement. These effects can be reduced by the use of 'soft' bearings and/or by varying the spacing of bearings.[5.24,5.40]

In continuous skew slabs with large width-to-span ratios, the magnitudes of the moments do not greatly exceed those in equivalent continuous rectangular slabs. In narrow continuous skew slabs, the effect of the support arrangements on the moments and support reactions is not much more pronounced than for simply supported skew slabs.[5.24] Where circumstances permit, the designer should investigate the alternative possibility of replacing skew slab decks with rectangular decks as the cost of the resulting increased span lengths may be more than offset by the savings due to the simplicity of deck reinforcement and smaller abutments.

Rectangular slabs can conveniently be reinforced in the directions of the principal moments. In skew slabs, this requires fanning of the reinforcement to approximately coincide with the varying directions of the principal moments. This is not always practical. It might be advantageous to place the reinforcement parallel to the free edges and support lines or to use a skew or orthogonal mesh,[5.24,5.40] but this would result in an increased amount of reinforcement. Where the directions of reinforcement differ significantly from the directions of the principal moments, these have to be transformed to determine the amount of reinforcement.[5.40,5.42]

The effect of prestressing slabs can be simulated by loading the slab with the vertical components of the radial forces due to the curvature of the prestressing cables. The effects of axial force and, where applicable, the eccentricities are then considered separately (see Chapter 4, Section 4.4.2).

Where a deck slab is restrained at its boundaries against lateral

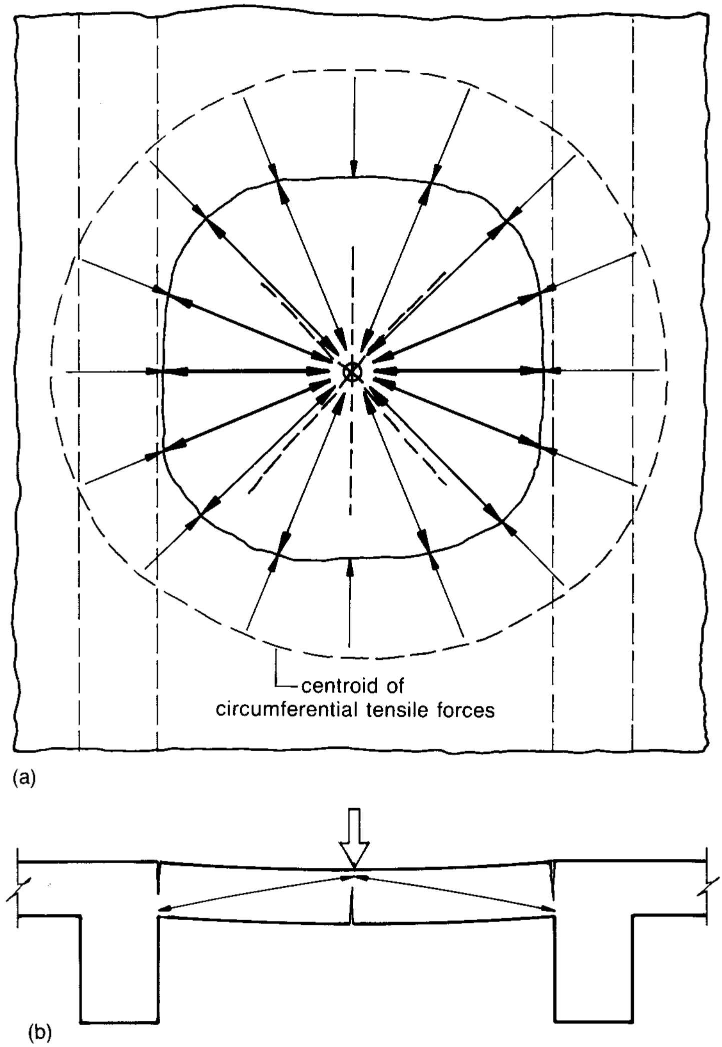


Figure 5.25
(a) Plan of slab showing distribution of compressive arching and circumferential tensile forces; (b) section through slab showing arch action resisting a point load

movements, arching action or the dome effect (also referred to as compressive membrane action) can greatly increase the ultimate limit of resistance against short-term loading (Figs 5.25a and b). This effect must not be confused with the membrane effect referred to by Hambly (Ref. 5.3, p. 139), which applies to closely spaced or contiguous beams where concentrated loading on one area of the deck results in a transfer of in-plane shear across the slab strips between longitudinal beams. The resultant antisymmetric shear flow results in in-plane bending of the slab. Because of the restraint of the surrounding slab area, the edges remain virtually straight and the slab distorts in trapezoidal shear. The transfer of in-plane shear also subjects the longitudinal members to axial loads, which in turn result in the neutral axis of the longitudinal beams moving up in regions of deck subjected to load and downwards elsewhere. The load-free deck to the side consequently behaves in membrane action like a very wide flange. This effect is induced in the purely elastic phase whereas arching action is induced after hair-crack formation in a concrete slab panel[5.34] and increases as yield lines in the slab develop due to the thrusting effect caused by the rotation of the slab elements bounded by these yield lines (Fig. 5.26). In lightly reinforced slabs a small degree of arching could, however, be present at an early stage to the extent that the compressive modulus of elasticity of concrete is higher than the tensile modulus. These effects are nor normally taken into account in the design of bridge deck slabs but could well be included and are useful when assessing an existing structure against overloads. Arching action is most effective in slabs with low percentages of reinforcement, of the order of 0.3 per cent. Ultimate loads more than four times the ultimate bending resistance have been recorded in laboratories and full-scale load tests. Serviceability requirements must however still be complied with, so that excessive cracking does not occur. Little is known about the fatigue behaviour of arch action but laboratory tests have shown that high stresses due to repetitive loads would reduce the effectiveness of the action because of the accumulative non-elastic portions of the deformations. A moving-wheel load could also cause a complex pattern of hair-cracks. In cases where an infrequent ultimate load is the critical consideration, the procedure is more relevant. A considerable literature has been published on the subject.[5.34,5.43,5.44]

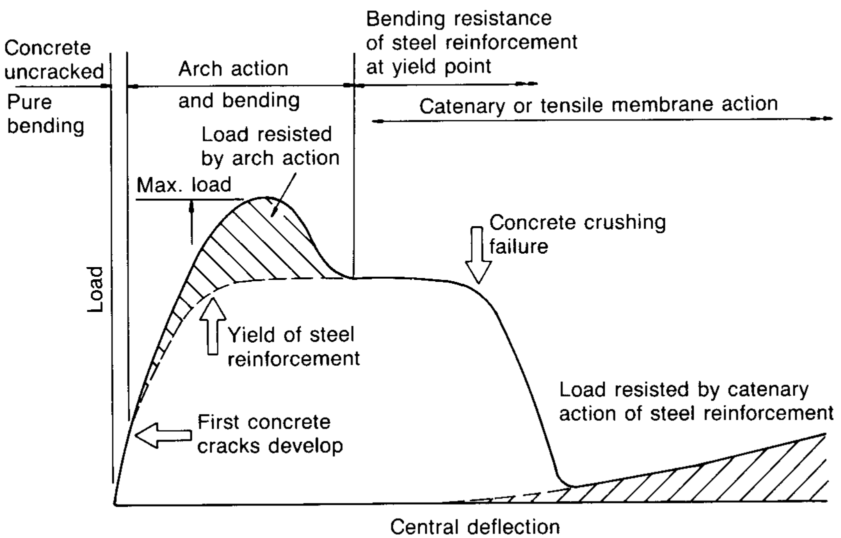


Figure 5.26 Load–deflection curve for a reinforced concrete slab in which arch action occurs

5.5.2.2 Hollow Slabs

When the required depth and resulting dead load become excessive, slabs are usually made hollow. Commonly used void formers made of polystyrene, hollow reinforced cardboard, compressed hardboard, sheet metal, timber, inflated rubber tubes, etc. are cast into the deck to form voids of circular or rectangular cross-section or as required (Fig. 5.27).

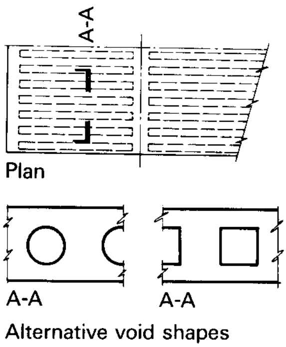


Figure 5.27
Voided slab

Longitudinally, the stiffness and strength are not materially affected. The transverse stiffness is, however, reduced. The exact reduction depends on the shape and size of the voids, the width of the ribs and the thickness of the top and bottom flanges. According to Hambly[5.3] if the width and depth of the voids is less than 60 per cent of the overall width and structural depth respectively, the slab still behaves like a plate. Hollow slabs are usually provided with crossbeams at least at the supports and at midspan for effective transverse distribution of moments and shears. If the voids become large, the slab acts like a cellular deck, i.e. deformations due to 'shear' become important and the deck no longer behaves like a simple plate.

5.5.2.3 Coffered Slabs

These act like a grid and are suitable for decks where the moments are primarily sagging (Fig. 5.28).

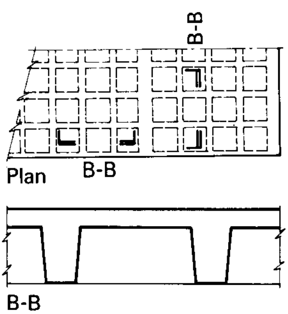
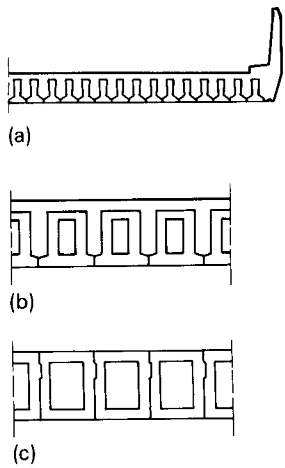


Figure 5.28
Coffered slab

5.5.2.4 Precast Beams

Where it is inconvenient or impossible to erect staging such as over existing roads or railways, precast beams can be laid contiguously next to each other. Plate action is usually achieved by casting *in situ* concrete strips between the beams with or without a thin *in situ* top slab. The slabs are transversely reinforced or post-tensioned. Ingenious methods of achieving 'dry joints' have been developed such as the Dywidag 'contact method'.[5.45] Figures 5.29(a)–(c) show various cross-sections of precast slab construction.

Figure 5.29 Precast beam slab decks: (a) T-beams with *in situ* concrete topping; (b) 'tophat' beams with *in situ* topping; (c) contiguous beams



5.5.3 Beams

A traditional form of superstructure consists of precast I- or T-beams, with precast or *in situ* crossbeams and deck slabs. In this form, the primary structural elements act as a grillage, the deck slab spanning between beams and acting compositely with the beams as top flanges. Crossbeams are usually provided at the supports and at the one-third points of the span. Figures 5.30 and 5.31 show two versions. Unless

Figure 5.30

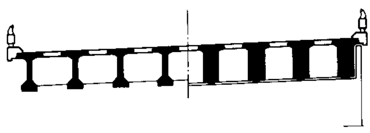
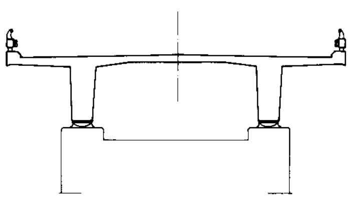


Figure 5.31



post-tensioning is applied where precast elements are joined together, the gap provided for *in situ* concrete must be sufficient to provide adequate bond for the overlapping reinforcement.

Precast post-tensioned I-beams have a useful span range of 20–35 m (66–115 ft) whereas precast post-tensioned T-beams are suitable for span ranges up to 45 m (148 ft). The main limitations are transport and handling problems.

With the demand for speed of construction and the widespread increase in labour costs, simplicity of construction has become ever more important and crossbeams have become less popular. Various deck profiles suitable for *in situ* casting on travelling formwork have become popular. These include single-spine or twin-beam systems, usually without crossbeams; see also Section 5.6 and Fig. 5.32. In the case of twin-beams, crossbeams should, however, be provided at the end supports both for torsional fixity of the beams and to keep the slab moments and deflections at the ends within acceptable limits. In the span and over intermediate supports in continuous structures with twin- or multi-beams, the slab can be designed to provide transverse stability by interaction in bending with the main beams. Torsional resistance of the deck system in these cases is resisted by a combination of torsion in the beams and a warping action of the beam and slab system. Figure 5.28 shows a typical cross-section.

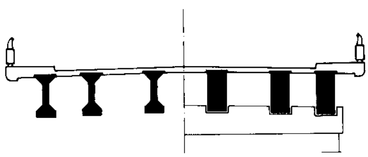


Figure 5.32

The design of beams is covered in Chapters 12, 13, 16, 19 and 20 of Ref. 5.1 and in most national codes (see Chapter 3) and will not be repeated here; see also Ref. 5.24.

Upstand beams are used in through bridges where maximum clearance is required. Cross-sectional shapes commonly used are rectangular or I-beams. They are not aesthetically very pleasing and are only really suitable for narrow structures. Figure 5.33 shows a pedestrian bridge across a railway line where the authorities required solid balustrades at least 1.450 m ($4\frac{3}{4}$ ft) high, which were used as beams.

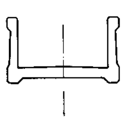


Figure 5.33
Pedestrian bridge with upstand beams

5.5.4 Box-Girders

Box-girders are extremely versatile and suitable for many applications. Their principal advantages are:

1. The relatively large torsional stiffness of the closed section.
2. Good sectional properties for bending resistance. The difference between the upper and lower section moduli at any section is not as great as for a T-beam slab deck. This results in less creep deformation in prestressed structures, and the lower flange provides a larger compression zone at the supports and more space to accommodate the tensile reinforcement at midspan.
3. Numerous arrangements of box sections are feasible to suit any deck width. The slab thickness and corresponding web spacing can be varied to suit (see Chapter 1, Section 1.3).
4. Its bending and torsional stiffness makes it suitable for bridges curved in plan.

For the above reasons, box-girders are eminently suitable for long spans. The main span in an alternative proposal for the Great Belt Bridge is 325 m (1066 ft) long.[5.11]

The box section can be constructed as a single or multicellular box, although the tendency for large bridges is to use a single cellular box with large cantilever deck slabs which can be stiffened with ribs or struts, for example the Eschachtal Bridge[5.20] (see also Fig. 5.34).

In addition to simple beam action, single-box sections and twin- or multiple-box sections, interconnected by slab decks, are subject to distortion and warping. The effects of shear lag, distortion and warping are shown diagrammatically in Fig. 5.35. As with any beam and slab system, shear lag effects may have to be considered in the flanges of box-girders, including the cantilever deck slabs. In regions where temperature gradients between the inside and outside of boxes may be large (> 20°C, 36°F), ventilation should be provided. Box-girder reinforcement should be adequately detailed to ensure the integrity of the webs and slabs.

For the analysis of box-girders, see Chapter 4 Refs 5.3 and 5.46; for reinforcement and prestressing, see Ref. 5.24.

5.6 Arch Bridges and Related Types

5.6.1 General

Various configurations of arch-type bridges are summarized in Chapter 1, Section 1.3. For reasons of space, it is not possible to discuss all these types in any detail. This section will therefore deal mainly with the

Figure 5.34 Hammersmith Flyover, London (courtesy of Maunsell and Partners)

modern form of concrete arch bridge, namely, the open arch with spandrel columns. The strut-frame type is described in Ref. 5.47.

Before the advent of the computer, arch bridges were designed using classical theory based on the principles of elasticity and the theory of small displacements. In order to reduce the considerable effort in computation that was necessary to solve the equations which govern the elastic behaviour of an arch, various assumptions and simplifications to the structure and the theoretical modelling thereof were usually made.[5.48]

With the development of modern computing methods and a clearer understanding of the reliability of structures, it has become practicable to simulate the total structure and the various actions and effects thereon much more realistically, with the result that it is now possible to design concrete arch bridges to closer tolerances with greater reliability. This, together with the efficient use of materials with improved strength properties, has led to slender structures, making it possible to eliminate hinges and to improve the economics of arch bridges in general. In concrete, other bridge configurations were, however, preferred in the 1950s and early 1960s because of the high cost of centering required for arch construction. Some engineers predicted that concrete arches

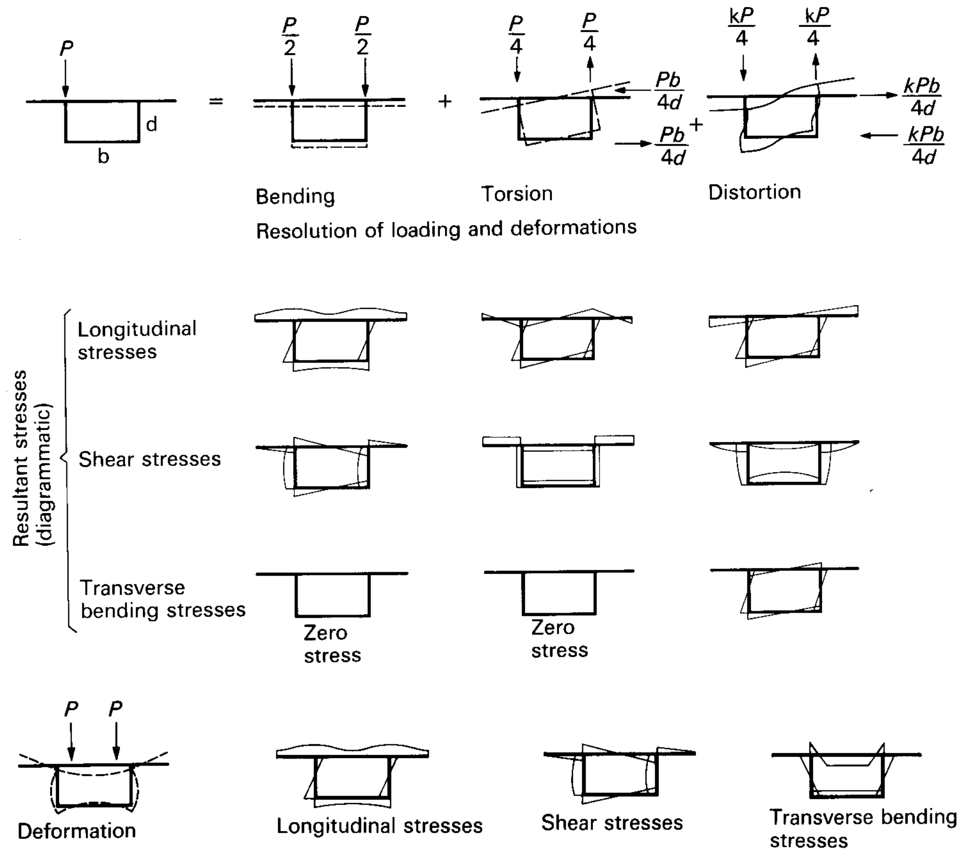


Figure 5.35 Deformations and stress patterns (diagrammatic)

would be superseded by other more economical structures such as cable-stayed girder bridges, even at sites ideally suited to arch construction. However, the development of the suspended and truss cantilever methods once more made concrete arches competitive up to spans approaching 400 m (1300 ft); see Chapter 1, Section 1.2. These methods are described later in this subsection.

5.6.2 Structural Configuration of Concrete Arch Bridges

Modern medium- to large-span concrete arch bridges appear to have optimum characteristics in the form of open spandrel arches composed of the following members (see Figs 1.11, 1.12 and 1.13):

1. A continuous prestressed or reinforced concrete deck supported by the abutments and at regular intervals by spandrel and approach columns, which are usually also dimensioned to provide lateral support to the deck.

2. A reinforced concrete arch which, on large arch bridges, is usually designed as a hollow box section with full fixity at the springings. Solid arches may, however, be more economic on shorter spans. Although hinges are normally avoided, they could be beneficial under specific conditions for shorter spans.
3. The proportioning and arrangement of the above members are interdependent because of their interaction in resisting various actions.

The following factors require careful consideration.

5.6.3 The Influence of the Site on the Dimensional and Geometric Requirements

On a major bridge structure, especially arch bridges spanning gorges, the exact horizontal and vertical alignments of the bridge should not only be located in terms of road geometric requirements. The total cost is determined by the dimensions of the structure, as well as by the nature of the approaches and the foundation conditions. It is therefore essential that a detailed foundation investigation be conducted before the structural configuration is finalized. The level of the roadway is an important factor that can influence the configuration. It will, for instance, affect the span-to-rise ratio of the arch, the height of columns, the position of abutments and therefore also the length of the deck. All these factors influence the overall economics of the structure, which should be balanced against the cost of cutting or fill for the approach roads (see Section 5.2).

The foundations of the arch are parts of the primary support system and both should preferably be at the same elevation. The positions of the foundations of the approach columns must be carefully considered and investigated, especially on steep banks, to ensure stability.

Where use is made of the suspended cantilever method for construction of the arch, suitable anchorage conditions for the tie-back system must be available. This requires sound rock formations.

The width of the deck is an important structural feature and is determined by the number of carriageways, traffic lanes, shoulders and sidewalks. It affects the relative transverse arrangement of deck members and columns, and therefore the cross-section of the arch, all of which contribute to the lateral stability and flexural stiffness of the structure and affect the economics of the bridge (Fig. 5.36). Future widening of the deck may need special consideration in the location of the bridge and in the design and arrangement of the supporting elements. For dual carriageways, consideration must be given as to whether the bridge should be separated into two parallel structures or combined in one structure. On long-span arch bridges there are normally structural and economic

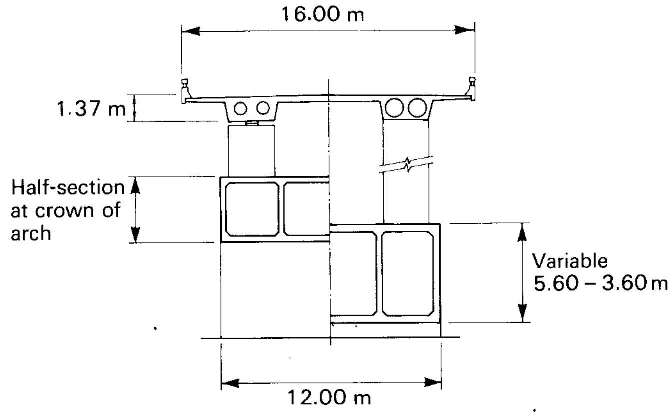


Figure 5.36 Section through arch and deck (Bloukrans Bridge, South Africa)

advantages in a single wider structure with greater transverse stiffness than two separate structures.

The shape of the arch should preferably be symmetrical with respect to the crown of the arch in order to avoid permanent displacement or sway in the longitudinal direction of the bridge. A difference in the number of spans on the embankments and hence the length of the deck on both sides of the arch due to an unsymmetrically shaped valley or river bed is normally not a problem, provided that the deck is symmetrically supported relative to the crown of the arch, and provided that adequate provision is made for the effects of longitudinal displacements in the bearings and columns.

With an unequal number of spans on the embankments, however, transverse bending of the deck due to wind, earthquake and eccentric live load will not be symmetrical about the crown of the arch. An uneven distribution of loading along the structure must be considered and carefully analysed, especially the displacement and rotational effects on the columns and bearings (see Fig. 5.39).

5.6.4 Design Aspects

5.6.4.1 Foundations

The foundations of an arch bridge should be able to resist the thrust from the arch without excessive settlement in any direction. It may be feasible to improve the load-bearing characteristics of the rock by the injection of cement in the highly fissured zones. Consideration must be given to the construction stages when large eccentricities of the thrust may result in high edge pressures. On large arch bridges, it is most important to establish the upper limit of probable settlement of the foundation. This can be estimated by means of two- or three-dimensional displacement

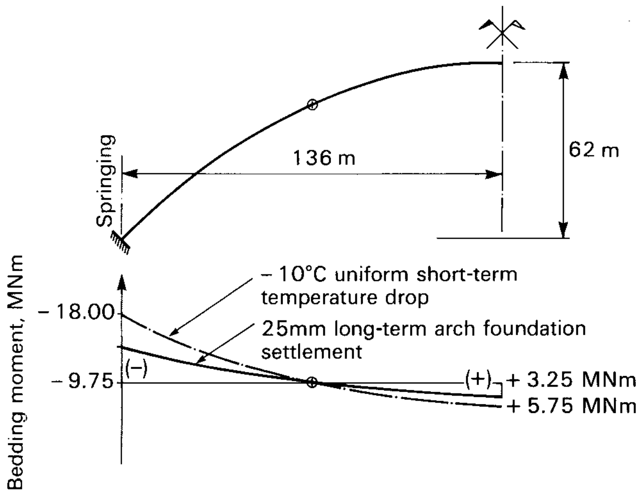


Figure 5.37 Typical effect of 25 mm long-term arch foundation settlement compared with unit temperature effect (Bloukrans Bridge, South Africa)

analyses in which the properties of the rock mass, joints and other discontinuities are taken into account.[5.49–5.51] The amount of foundation settlement that can be accommodated by the arch will depend on the shape, the sectional properties and flexural stiffness of the arch and the quality of concrete and amount of reinforcement (Fig. 5.37).

5.6.4.2 Columns

Structural considerations dictate that the columns should be as light and slender as possible, especially on the arch.where the concentrated dead-load reactions of the columns result in long-term local peak bending moments in the arch (Fig. 5.38).

Due to the relatively large longitudinal movements of the continuous deck when subjected to temperature changes, it is advantageous if the columns are made flexible in the longitudinal direction of the bridge, thereby reducing bending moments in the columns and, in the case of the taller columns, making hinges or sliding bearing unnecessary. Slender columns are also beneficial for providing the necessary flexibility in order to reduce the effects of earthquakes (see Chapter 4, Section 4.6). If the deck is subjected to ground accelerations, large inertial forces will result from its considerable mass. Rigid connections restraining movement between the deck and the foundations should therefore be avoided. All connections between the deck and the columns and other supports should be designed to accommodate the maximum relative displacements that may develop during seismic disturbances.

Care should however be taken to ensure that the structure as a whole has adequate flexural stiffness and strength in the longitudinal direction to withstand the effects of longitudinal wind action, braking and

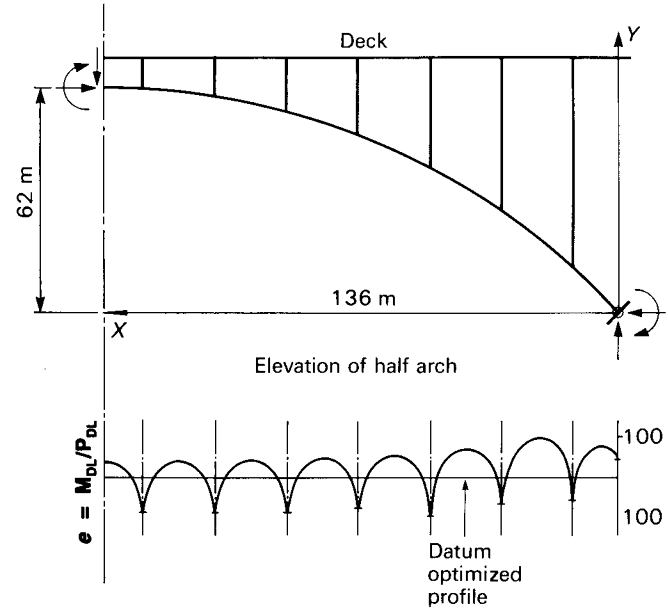


Figure 5.38 Eccentricity M_{DL}/P_{DL} in arch due to dead-load effects at $t = \infty$ (Bloukrans Bridge, South Africa)

acceleration forces due to traffic. The required stiffness of the columns in the transverse direction of the bridge will depend on the relative stiffness of the deck and arch. Under the action of transverse wind loads, earthquake effects and eccentric traffic loading, the deck acts as an horizontal girder spanning between the abutments and interacting through the columns with the arch, which behaves like a bow-girder (Fig. 5.39). In determining the minimum column dimensions, stability against buckling must be considered. This may require non-linear analyses of the individual members or the structure as a whole, as described in Chapter 4 Section 4.5. The spacing of columns is an extremely important factor which must be carefully investigated. The normal principles governing the design and economics of girder bridges generally apply to the approach spans, but additional factors have to be considered for the spans over the arch. The optimum spacing of the spandrel columns will not only depend on the economics of the deck system and that of the columns, but also on the influence of the spacing of the columns on the cost of the arch. The final spacing selected should, however, also satisfy the basic structural requirements for continuous deck systems. It is aesthetically preferable to have a column on each side of the crown and not *on* the crown of the arch. For constructional and economic reasons, it is advantageous to position columns on the arch foundation — preferably behind the springing. This generally improves the stability of the arch foundation and eliminates two sets of column foundations.

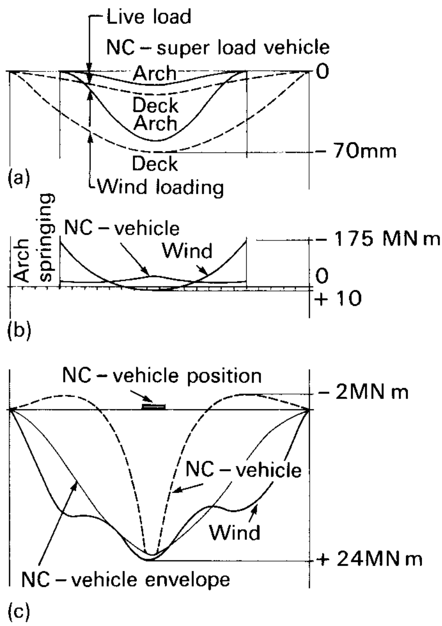


Figure 5.39 Transverse behaviour of the Bloukrans Bridge, South Africa: (a) transverse displacements; (b) transverse bending moment in arch; (c) transverse bending moment in deck

5.6.4.3 The Arch

The economy of an arch is related to the profile which, if correctly designed, will ensure that the selfweight of the structure induces mainly compressive forces. This requirement also ensures that the deformation of the arch due to creep of the concrete is limited to mainly axial shortening of the arch rib which does not give rise to large secondary forces in the rest of the structure. This is achieved by making the arch shape correspond as closely as possible with the line of thrust due to the dead loads, thereby minimizing the eccentricity of the thrust on the arch.

From an aesthetic point of view and for constructional reasons, it is preferable to have a smooth-arch profile. The above requirement can therefore only be partially satisfied by selecting a smooth profile which results in minimum deviations between the centroid of the arch and the line of thrust. On large arches, the most suitable profile is normally a power function which is fitted to equalize the positive and negative areas of the bending moment diagram along the arch rib, thereby minimizing permanent deflection of the arch rib (Fig. 5.38).

Other long-term effects are shrinkage of the arch rib and foundation settlement of the arch springings. Provision can be made to counteract the long-term stress effects due to creep and shrinkage by constructing a predetermined profile which would initially set up selfweight bending moments approximately equal but of opposite sign to the moments due to creep and shrinkage. Provision in the arch profile to reduce live-load moments is not recommended. Due allowance should also be made to allow for deformations that would result from the construction procedure in order to achieve the required profile in the long term.

On slender concrete arches the non-linear effects due to the deformations of the arch under load ($P-\Delta$ effects), as well as the non-linear strain effects on concrete when subjected to high compressive stresses, should be investigated. The resistance against buckling under heavy eccentric loads should be calculated by means of a non-linear analysis up to failure, considering both geometric and material non-linearity of the whole structure.

The actual dimensioning of the arch cross-section and reinforcement is strongly influenced by the construction method.

5.6.4.4 The Superstructure

The deck structure is usually designed as a continuous member supported by columns, with expansion joints at each end. Concrete decks on arch bridges may be in prestressed or reinforced concrete, and may be constructed by various methods, as applicable to slab, beam or girder bridges.

The degree of flexibility, and hence deformation of the arch which supports the deck, will increase the moment range at any particular section due to the various combinations of loads. This may result in a system where reinforced concrete is more economic than a fully prestressed section. This will, however, depend on the stiffness of the arch, the relative stiffness of the deck, the column spacing and the type of superimposed live loads. The spans on the banks are less highly stressed than the spans over the arch except over the first column of the arch, which is supported directly on a bank, where high moments can develop due to the deformation of the arch. The transverse forces on the deck and the resultant behaviour have already been described.

The design of the deck is influenced by the construction method, as is normally the case with all continuous deck systems constructed in stages. The redistribution of moments in the structure because of the creep effects on the concrete in the main structural elements, namely the deck, the columns and the arch, must therefore be considered.

5.6.5 Analytical Procedures

Chapter 4 describes the relevant analytical methods and gives references for further details. In long-span arch bridges the following systems have to be analysed.

5.6.5.1 Analysis of the Construction Stages

The construction stages of an arch bridge may be divided into the following main stages:

1. Construction of the foundations, approach span columns, abutments and approach deck spans.
2. Construction of the arch.
3. Construction of the spandrel columns and deck spans over the arch.

Depending on the construction techniques adopted for the arch and the deck, these stages may be executed in sequence or may be combined.[5.52,5.53] During the construction stages, the structure undergoes various structural changes during which bracing and staying may be required until the permanent structural system is completed and the bridge becomes self-supporting. It may be necessary to analyse a large number of these structural systems at the construction stages for the load effects specified by the applicable codes of practice. Temperature effects on suspension cables are important.

5.6.5.2 Analysis of the Completed Structure

An analysis is made of the completed structure subjected to:

1. Long-term dead-load effects resulting from the construction stages. This analysis should include long-term effects developing from creep, prestress, shrinkage and support settlement and should be an extension of the stage-by-stage analysis of the bridge covering all the construction stages.
2. Traffic loading as specified by the relevant design codes. For this analysis, the completed structure is normally simulated by a space frame with short-term elastic stiffness properties.
3. On slender members, the effect of non-linear material properties as well as geometric non-linearity of the loaded structure should be analysed for the post-elastic stress range up to failure to investigate instability due to buckling when subjected to unsymmetric loading (see Chapter 4, Section 4.5). This analysis may comprise the whole structure as a two-dimensional frame or individual members with slender proportions (see Chapter 4, Section 4.5).
4. Temperature effects including axial and gradient temperature effects for the serviceability limit state of cracking. (Restraint forces due to temperature effects need not be considered for the ultimate limit state.)
5. Wind loading as prescribed by the relevant design codes for the particular area. On longer slender arch-type bridges, dynamic or

semi-dynamic analysis may be necessary using a space-frame simulation.

6. Earthquake or seismic effects as prescribed by the relevant design codes for the particular area.

5.6.6 Planning and Design of the Construction Stages of Arch Bridges

The construction of the arch by means of the various cantilever methods requires careful preplanning with special bracing and staying to be provided for the safe erection of the arch.

It is not in the scope of this book to deal with the details of the various alternative construction methods and the reader is referred to specialized literature dealing with specific projects.[5.52–5.57]

The suspended cantilever and cantilever truss construction methods employed on some large arch bridges have been mentioned in Chapter 1, Sections 1.2 and 1.3. A brief résumé of these methods will be given here with reference to the construction of the Bloukrans Gorge Bridge.[5.57]

5.6.6.1 The Suspended Cantilever Method[5.47,5.55–5.57]

The arch is constructed in stages from both embankments using travelling formwork carriages on which each new segment is constructed. The travelling formwork carriages are supported from the previously completed segments and are moved forward for the next construction stages once the concrete has reached the required strength. As the cantilevered portions are extended, they are supported by suitably placed suspension cables which are stressed to predetermined forces in order to provide the required support to maintain the correct arch profile and bending moments at each particular stage (Fig. 5.40). The suspension cables are supported by temporary towers which are tied back and anchored into the rock. A construction cycle of approximately 7 days per segment can be achieved.

Temperature effects on the cables are monitored and allowance is made in the forces to which the cables are stressed to allow for any temperature deviations above or below a chosen datum temperature.

The main construction phases can be summarized as follows:

1. The suspended cantilever construction stages for each segment of the arch.
2. The profile adjustment prior to closure of the arch (see Fig. 5.41).
3. Closure of the arch (see Fig. 5.42).
4. Removal of the arch carriages.
5. Removal of the suspension cables.

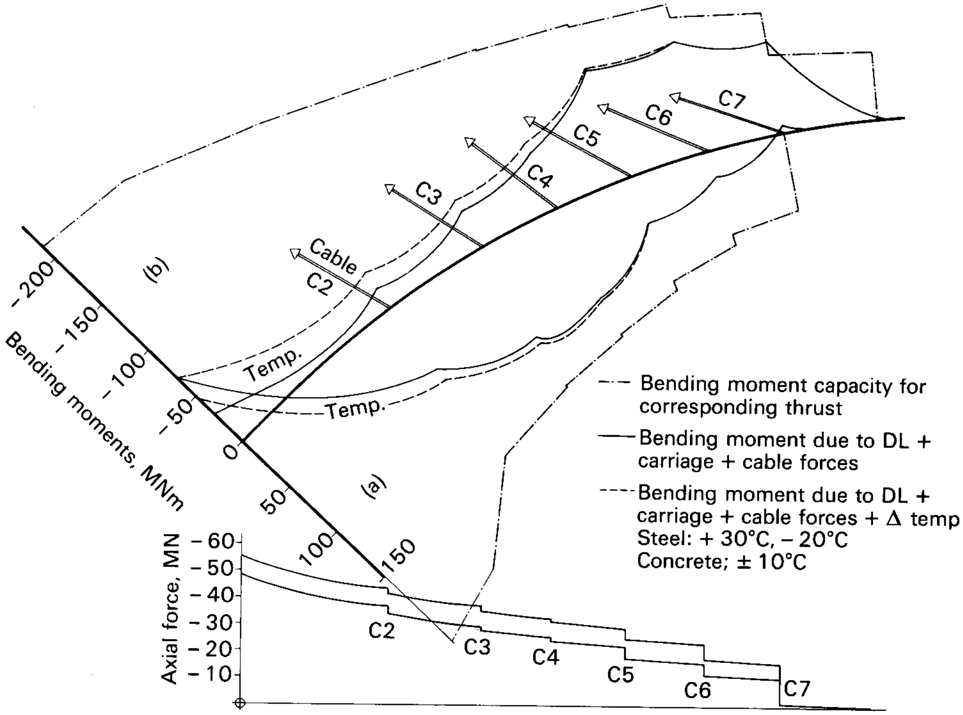


Figure 5.40 Arch cantilever construction, construction of segments 25 to 27: (a) DL segments 1 to 24, cable C7 tensioned; (b) DL segments 1 to 27. · — · — · — · — · Bending moment capacity for corresponding thrust; ——— bending moment due to DL + carriage + cable forces; — — — — bending moment due to DL + carriage + cable forces + Δ temp. Steel: +30°C, −20°C. Concrete: ±10°C.

Each of these stages has to be preplanned to follow a detailed procedure and analysed accordingly so as to obtain the correct profile of the arch at completion, allowing for subsequent creep and shinkage (see Fig. 5.43).

During construction it is normally attempted to utilize the steel reinforcement that is required for the permanent service condition of the arch. It may, however, be necessary to increase the arch reinforcement in certain sections to increase the bending capacity for the construction stages. Although these moments can be accurately controlled by the regular adjustment of the cables, the effort and cost required for the additional adjustment of cable forces must be evaluated against the cost of extra reinforcement. The closure procedure involves the temporary locking of the gap (for which various techniques are available)[5.57] to avoid overstressing the green concrete due to temperature effects followed by a gradual release of the remaining suspension cables as the concrete hardens.

Usually with this method, the deck over the arch can only be constructed after completion of the arch. It is therefore possible to re-use the suspension cables for the prestressing of the deck.

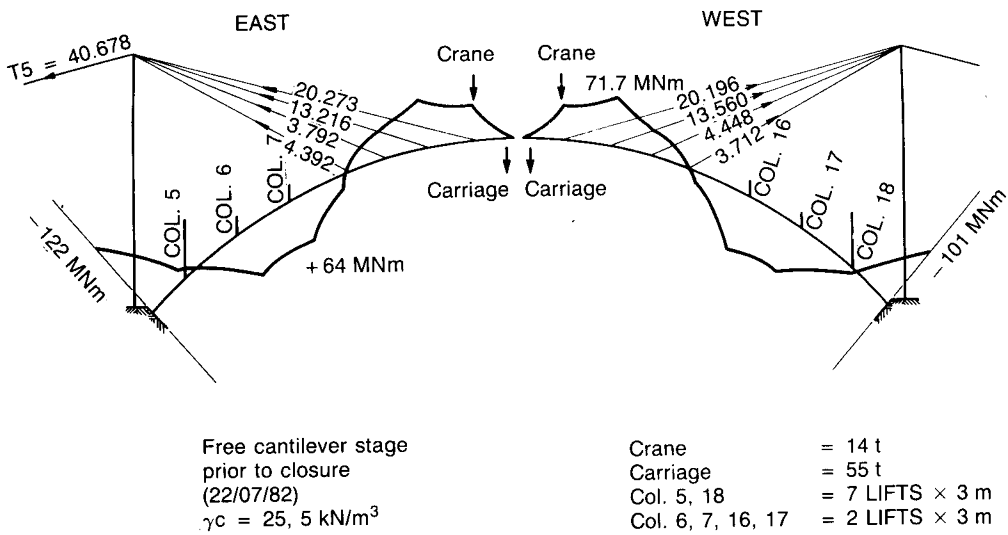


Figure 5.41 Dead-load bending moments prior to closure at crown

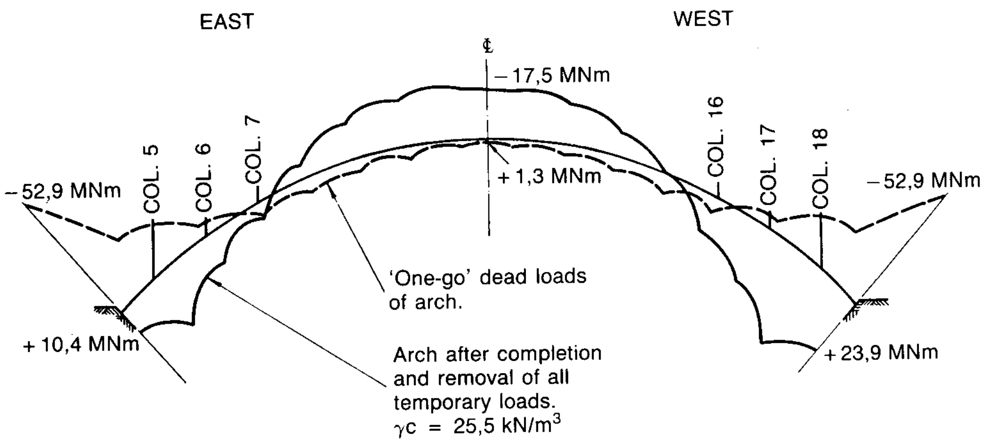


Figure 5.42 Dead-load bending moments in arch after completion

5.6.6.2 The Cantilevered Truss Construction Method

This method has been successfully used for some of the world's largest concrete arch bridges.[5.52–5.54] The method comprises the construction of the arch in stages as a cantilever truss in which the arch constitutes the main compression chord and the columns the vertical truss members. The temporary inclined ties and top tension chord members may be formed out of structural steel together with post-tensioned cables. The top tension chord member is extended and anchored back into the banks.

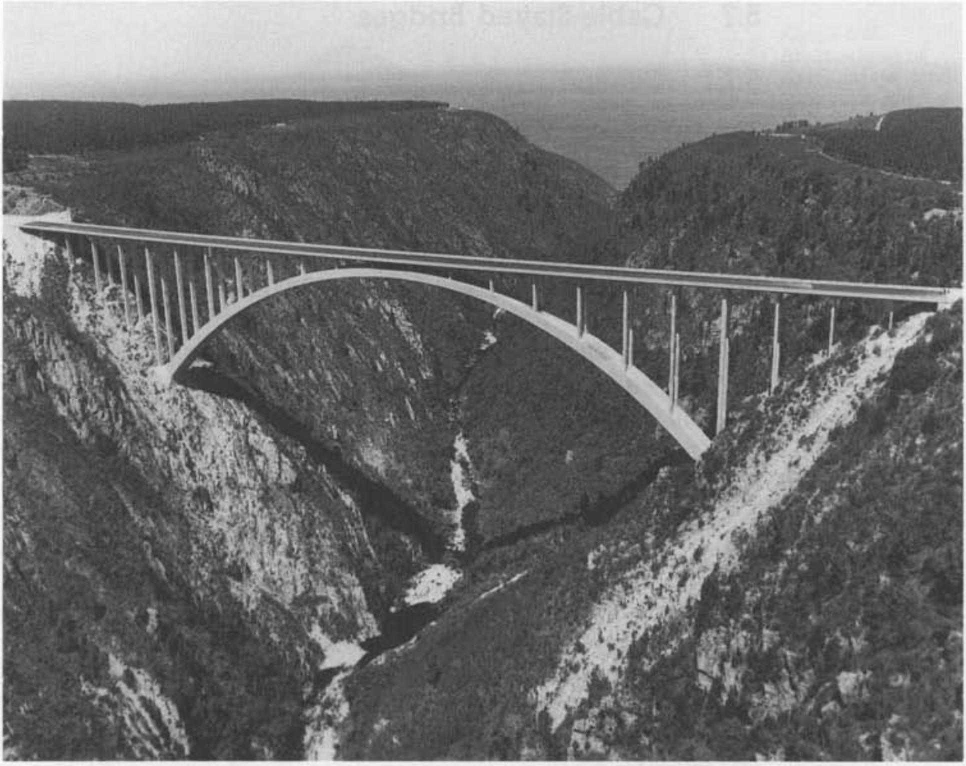


Figure 5.43
Bloukrans Gorge Bridge in South Africa

A further development of this method whereby the deck is constructed concurrently with the arch and spandrel columns has been applied successfully on a two-pinned arch bridge in Japan.[5.53] With this method, the inclined ties and main tension chord comprise temporary prestressing bars which are removed once the arch and deck have been completed (Fig. 5.44).

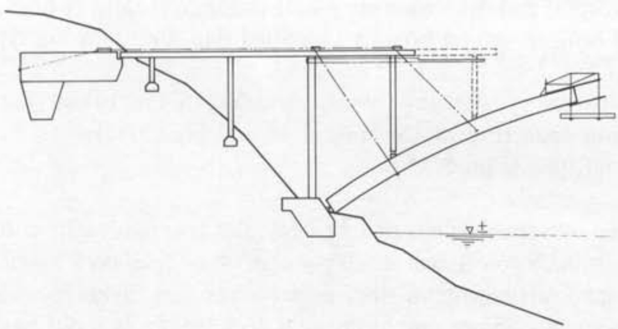


Figure 5.44
Cantilevered truss construction method (Hokawazu Bridge, Japan[5.53])

5.7 Cable-Stayed Bridges

5.7.1 General

The development of cable-stayed bridges is discussed in Chapter 1, Section 1.2, and the various structural configurations are summarized in Chapter 1, Section 1.3. Although most cable-stayed bridges have structural steel superstructures, reinforced and prestressed concrete has been successfully used in large spans of up to 320 m (1050 ft).[5.58] Proposals to extend the span range to 700 m (2300 ft) have been made.[5.24]

Because of the considerable scope for variation in the arrangement and design of concrete cable-stayed girder bridges, it is not possible to cover all types and details in the space available. However, some of the main aspects will be briefly dealt with and the reader is referred to specialized literature on cable-stayed bridges.

5.7.2 Main Structural Elements

The main structural elements are:

(a) foundations of substructures;
(b) substructures consisting of land abutments, main support piers, secondary support piers under side spans and approach ramps;
(c) superstructure;
(d) stay-cable pylons;
(e) stay-cables.

5.7.3 Span Properties of Cable-Stayed Bridges

Cable-stayed bridges are suitable for most sites but as the superstructures are supported from above by means of inclined cables they can be constructed without centering and are especially ideal for river crossings, deep gorges, and over wide busy traffic lanes and railway lines. Cable-stayed bridges can be broadly classified into the following types:

(a) two-span symmetric or asymmetric (Fig. 5.45a, b) and (Fig. 1.22);
(b) three-span (Fig. 5.45c, Fig. 5.46 and Fig. 5.48);
(c) multispan (Fig. 5.45d).

In the two-span asymmetric bridges, the average length of the main span normally constitutes ± 70 per cent of the total deck length. In the three-span arrangement, the centre span on average constitutes approximately 55 per cent of the total deck length. In multi-span cable-stayed bridges, all spans are normally equal except the end spans.

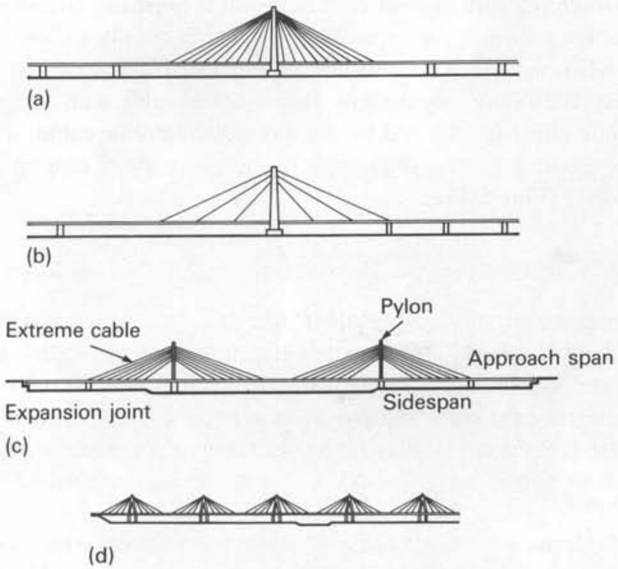


Figure 5.45 Arrangement of spans: (a) two-span symmetric with radiating cables (Ludwigshaven, West Germany); (b) two-span asymmetric with fan-type cables (Cologne, West Germany); (c) three-span with fan-type configuration (see Fig. 5.49); (d) multispan with radiating cables (Ganga, India)

The drop-in spans joining consecutive multispan stayed structures may comprise up to 20 per cent of the length of the total span between pylons for designs with single or few stay cables. In bridges with multi-cable-stays, the drop-in spans may reduce to approximately 8 per cent of the total span between pylons.

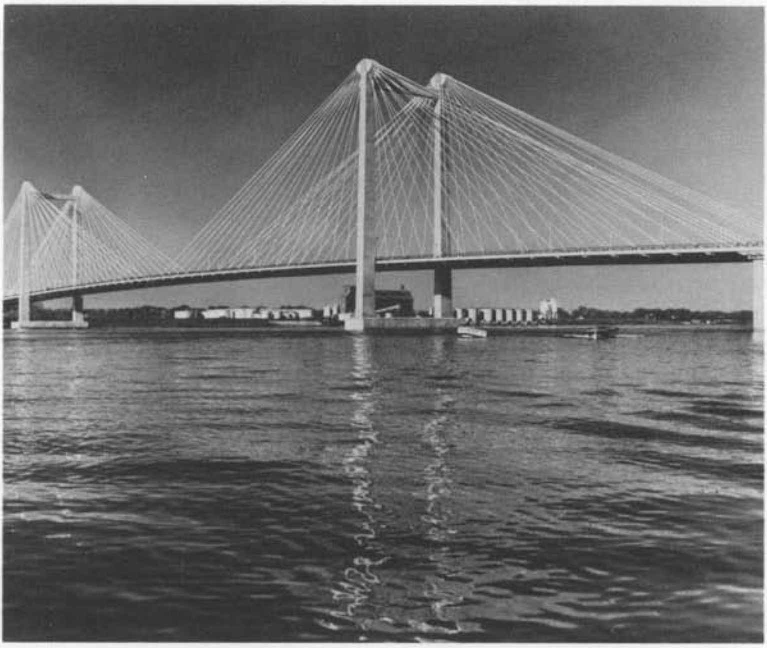


Figure 5.46 Pasco–Kennewick Intercity Bridge, USA (courtesy of Leonhardt, Andrä and Partner)

In structures with viaduct approaches, it is normally advantageous to anchor back stays at the approach piers: this normally stiffens the main span when subjected to unbalanced live loads, allowing a shallower section. With multistay designs, this is not possible with all cables but the same effect is achieved by anchoring the extreme cable, which has an increased area and load capacity, above a rigid support (pier or abutment) (Fig. 5.45a).

5.7.4 Geometry and Properties of the Cable-Stays

The number of stay cables, their size and their geometry may vary considerably for different bridges depending on the spans, the deck structural system and design loads. The arrangement of stays controls the design of the deck and pylons to a large degree, as mentioned in Chapter 1, Section 1.3. Stay cables may consist of parallel wire, parallel strands or ropes, single strands or ropes, locked coil strands or solid bars.[5.59]

In modern cable-stayed bridges, cables are normally constructed from standard strand or parallel wire. The wires are manufactured by the cold-drawn process, of specially heat-treated steel rods using successive dies to improve the internal structure of the steel and thereby the tensile strength properties. The ultimate strength of the wire may vary between 1400 and 1850 MPa [$(2-2.68) \times 10^6$ psi], depending on the properties of the rope, strand or wire.

Wires and cables are normally zinc-coated for protection to conform with a specific grade. Painting, rust-preventive compounds or plastic jacketing are used as added protection prior to and during erection. The mechanical properties of the finished product must be carefully controlled. Steel ropes and strands normally require pre-stretching to ensure true elastic properties throughout the stress range. Normally, pre-stretching up to 50–55 per cent of the ultimate strength of the ropes or strands is required to produce consistent elastic properties, which are essential in the design of the structure. Values for Young's modulus of elasticity (*E*-modulus) should be determined from long lengths of cable which will yield more reliable values than shorter test specimens. The values should be determined for a stress range between 10 and 90 per cent of the pre-stretching loads. The *E*-value varies with the type of cable, depending on whether it is strand, rope or parallel wires. The *E*-value is normally based on the gross metallic area of the cable including the zinc protection coatings and may vary from as low as 165 500 MPa (24×10^6 psi) for rope cables to 190 000 MPa (28×10^6 psi) for parallel wire cables. The allowable working stress for parallel wire cables is normally limited to approximately 50–55 per cent of the yield strength or 0.2 per cent proof stress.

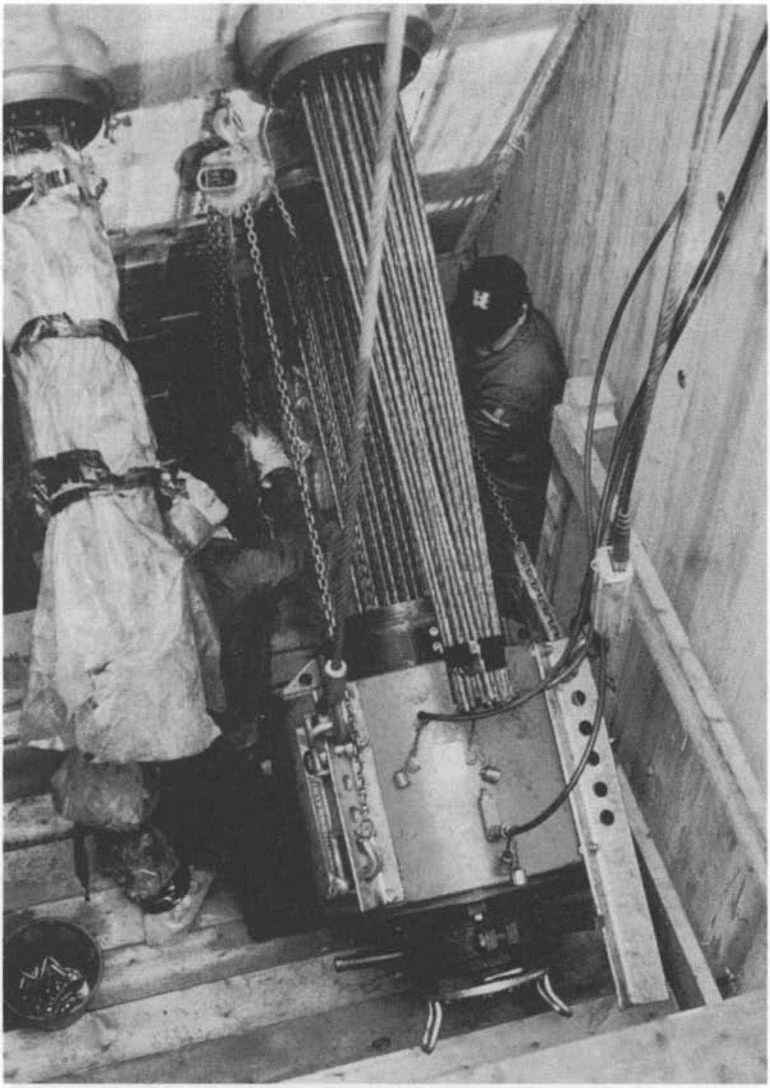


Figure 5.47 Cable jacks being positioned for Kemijoki Bridge in Finland (courtesy of Dr P Marti, VSL International)

End anchorages and saddles on the pylons must be designed to develop 100 per cent of the strength of the cables which they must anchor or support. Cables may be stopped off and anchored in the pylon or may be taken continuously through over saddles with adequate radius to avoid excessive bending stresses in the cables. Continuous cables are clamped to the saddles after tensioning to avoid sliding over the saddles.

The stay cables are part of the primary support system in a cable-stayed bridge and must therefore be protected against any type of corrosion. The most vulnerable sections are at the anchorages (Fig. 5.47).

While earlier cable-stayed bridges had cables that were encased in concrete (see Figs 1.8 and 1.22), various systems have since been developed for protection after installation, such as:

(a) multiple-layered plastic coating impregnated with glass-reinforced acrylic resins;
(b) elastomeric coating of liquid Neoprene, multiple coats of uncured Neoprene sheets and top coats of Hypolon paint;
(c) polyethylene or polypropylene ducts filled with corrosion-protective epoxy-enriched cement grout[5.60,5.61] (Fig. 5.48a);
(d) cement-grouted steel ducts.[5.62]

During the life of the bridge, the stay cables are subjected to a considerable number of stress fluctuations and must be protected against fatigue. The effects of vibrations due to wind must also be carefully considered and rectified if of significance. Apart from complying with fatigue resistant design criteria for the cables, special anchorages have been developed to limit stress concentration in the anchor zone of the wires, thereby considerably improving the fatigue-resistant properties of stay cables.[5.60,5.63] (Fig. 5.48b).

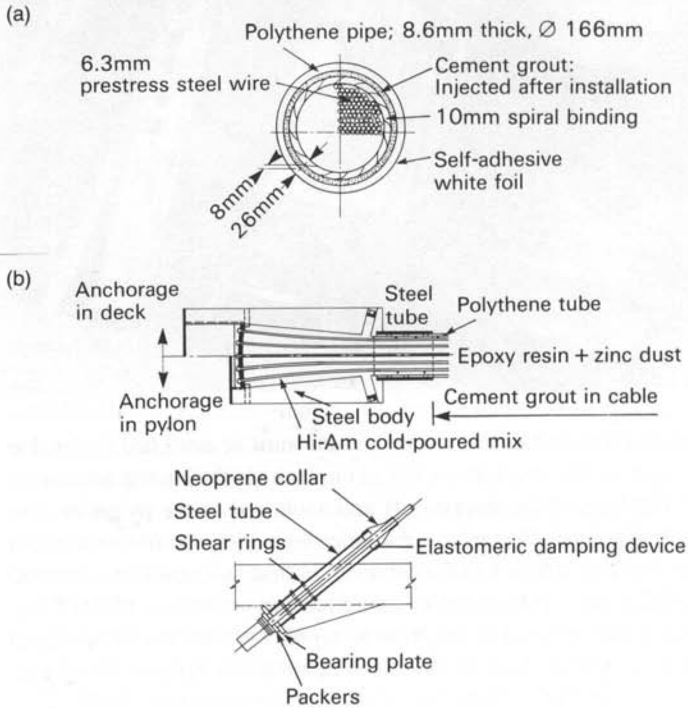


Figure 5.48 (a) construction of parallel wire stay-cable, and (b) cable anchorage in girder, (1) long section through head, (2) anchorage in girder, Pasco–Kennewick Bridge (courtesy of Leonhardt and Andrä)

5.7.5 Erection Procedures

Cable-stayed bridges may be erected by three basic methods: construction of centering, the push-out (incremental launching) method and the cantilever method.[5.59]

The construction of centering is generally used where the clearance between the superstructure and the underlying terrain is low enough to make temporary staging economically viable, where suitable founding

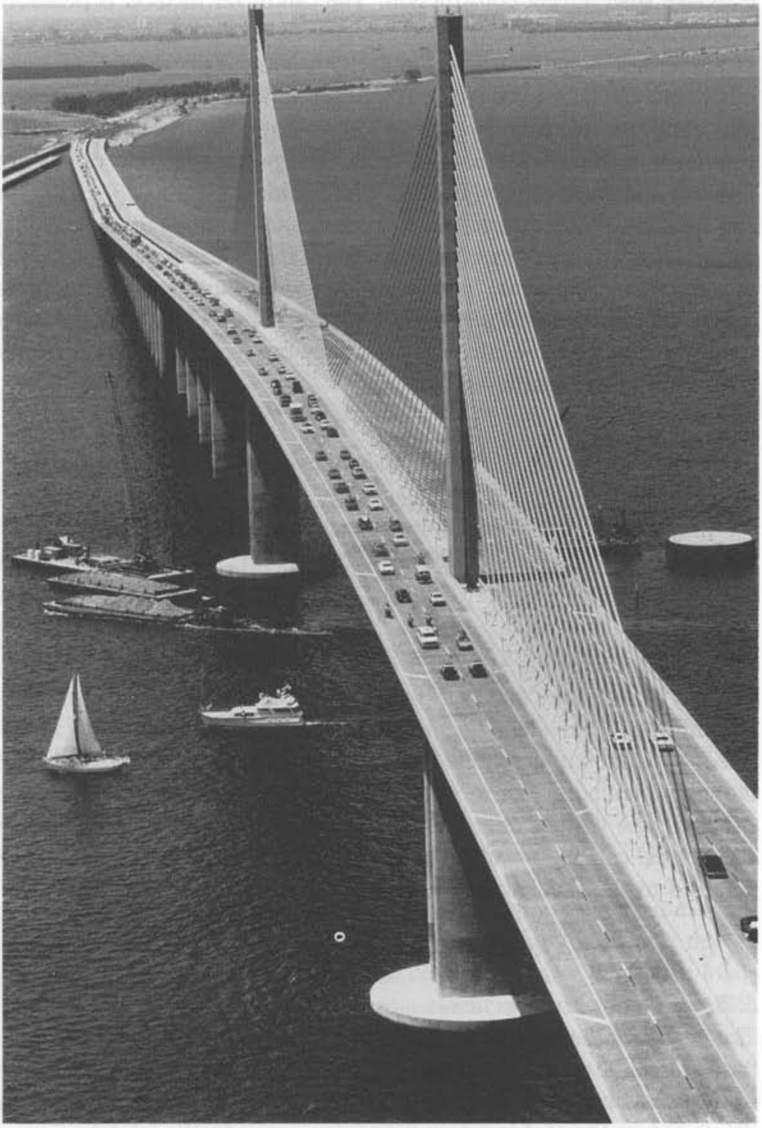


Figure 5.49 Sunshine Skyway Bridge, Florida, USA (courtesy of Dr P Marti, VSL International)

conditions exist, and where it will not interfere with land or water traffic or be exposed to dangerous floods. The incremental launching method has been used in some special cases where the cantilever method could not be applied because of inadequate clearances. With this technique, which is described in Section 5.3, the pylon with supporting stay cables, which form part of the primary structural system whether these are temporary or permanent, have to be launched together with the deck. Launching may require intermediate temporary supports to reduce the effective span during construction. The cantilever method is probably most often used with cable-stayed bridges, especially where the positions of temporary supports below the deck are limited or where traffic and other obstacles have to be crossed (see Figs 5.49 and 5.50). The technique may comprise free cantilevering or a combination of free and suspended cantilever methods (see Section 5.3). The permanent stay cables are utilized, sometimes in conjunction with additional temporary cables, to stay the cantilevered structure as it is constructed in stages using travelling formwork carriages or erection carriages in the case of precast segmental construction. Temporary piers may be used as intermediate supports where this is economically advantageous (Fig. 5.50). Designs with multiple stays have the advantage that they can be erected economically by means of the cantilever method.

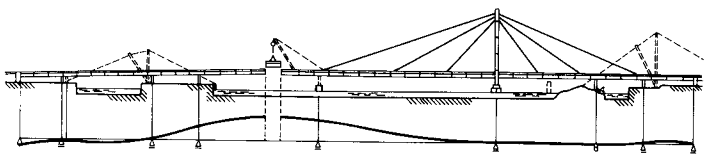


Figure 5.50 Free and suspended cantilever construction (*Stahlbau* **26**(4) April 1957)

During the erection stages, it is usually necessary to take the non-linearity of the cables due to sagging into account. Approximate formulae to allow for equivalent linear moduli of elasticity have been developed.[5.59] Concrete decks can be of *in situ* or precast construction and additional temporary prestressing may be required during the erection process. The various alternative possibilities are numerous, depending on the local conditions and the design of the bridge; the reader is therefore referred to the specialized literature.

5.7.6 Design Considerations

5.7.6.1 Deck Cross-Sections

The design of the deck cross-section is largely controlled by the spans, the width of the roadway and the longitudinal and transverse arrangement of the stay cables (see Chapter 1, Section 1.3). Alternative systems are illustrated in Fig. 5.51.

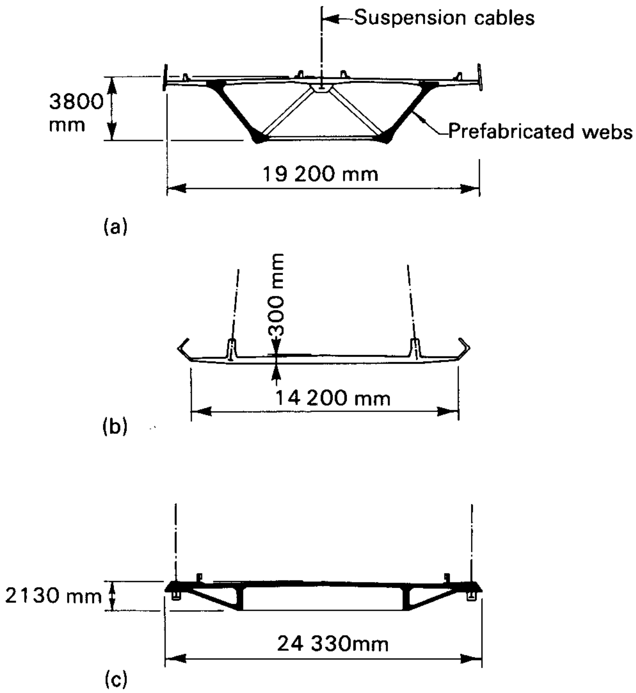


Figure 5.51 Typical cross-sections: (a) deck with precast web, Brotonne Bridge, France; (b) proposed cross-section for a narrow deck, design project; (c) deck in precast concrete, Pasco–Kennewick Bridge, USA

Designs with cables in a single plane require torsionally stiff girders, as illustrated in Fig. 5.51a. Systems with cables in two planes, which support the edges of the deck, can have relatively flexible and slender cross-sections (Fig. 5.51b). For wide decks, the slab may require intermediate transverse stiffening by means of transverse girders placed at regular intervals (Fig. 5.51c).

In a cable-stayed bridge where the stay cables are anchored in the deck side-spans, known as a self-anchored system, the stay cables cause compression in the main girder which, in concrete girders, is utilized as permanent prestressing of the deck. In this arrangement, expansion joints are normally provided at abutments (Fig. 5.45c). Additional prestressing may, however, be required near the end supports where the total effect of the horizontal components of the cable forces is less; this may cause local tension in the deck.

It is necessary to consider the forces in the superstructure during all stages of construction, making due allowance for the effects due to shrinkage and creep. The final stress condition under long-term loads can be controlled by precalculated adjustment of the suspension cables during installation. In concrete cable-stayed bridges, the secondary forces which develop due to creep are caused by two effects: one develops because of the relaxation of bending moments which are induced in the system and the other by creep shortening of the concrete members due to axial forces. Shrinkage will have effects in the axial direction similar to the creep shortening effects. The redistribution of bending moments can be controlled to be similar to that which develops in continuous systems on rigid supports, bearing in mind that an elastically supported system with moments which are equal to the 'one go' moments of a rigidly supported continuous girder will not be subjected to redistribution of bending moments due to creep. This implies that the moments in the girder of a concrete cable-stayed bridge will not change if the vertical components of the stay-cable forces are so arranged that they are identical to the 'one go' reactions of an equivalent rigidly supported continuous girder — if the axial shortening effects are not considered.[5.60]

Because of the creep and shrinkage shortening of the concrete members, bending moments develop in the girder and pylons similarly to the effects of a gradual support settlement in a continuous girder. It is possible, by means of adjustment of the stay cables, to induce bending moments in the girder of which the effects will be opposite and approximately equal in magnitude to those caused by the axial shortening due to creep.[5.60]

5.7.6.2 Non-linear Behaviour of Cable-Stays

In cable-stayed bridges there may be cases when some of the stay cables behave non-linearly because of the catenary effect of a cable under low tension. This effect is due to the change in the rate of increase of the tension force as the sagging profile straightens out, or vice versa. As the behaviour of the structure is dependent on the relative stiffnesses of the various structural elements, i.e. girder, pylon and cables, it will be affected by the effective stiffnesses of the cables, which depend on the initial pretension.

Normally, the structure starts behaving linearly at sag ratios of less than 1:80. During construction, when dead load is applied in stages, the cable forces build up from an initial low pretension. During these stages, the cables therefore behave non-linearly. It is desirable that when all dead loads have been applied, the 'initial sag ratio' should be well below this value of 1:80.

In large-span concrete cable-stayed bridges, the dead loads constitute the largest portion of the total load, resulting in relatively high pretension forces under dead load. Hence, the catenary effect in the cables when live loads are applied to the structure can, in many cases, be ignored, i.e. the structure behaves linearly during the application of live load.[5.59]

Depending on the structural configuration, the slenderness of individual members and the concrete stress levels, it may be necessary to perform non-linear analysis to investigate the stability of all members under loads approaching the required ultimate values.

5.7.6.3 Behaviour of Stayed Bridges under Wind Loading

The results of dynamic analyses of various cable-stayed structures, especially designs with multistays in two planes, have shown that cable-stayed bridges are aerodynamically far more stable than suspension bridges. This can be attributed to 'system damping' effects which come into play as soon as the amplitude of the dynamically induced displacements become large enough to be structurally significant. These 'damping systems' can be attributed to the following phenomena:

1. The non-linear spring stiffness of the cables which reduces with increased upward deflection of the deck because of a reduction in cable tension causing increased cable catenary. (Each cable also behaves differently dependening on its geometric properties.)
2. The interaction between the various individual systems, namely, the cables, pylon(s) and deck girder which all vibrate at different natural frequencies, resulting in interference of oscillations which prevents resonant conditions with large amplitudes of deflections developing.

Multi-cable-stayed concrete bridges (Figs 5.46 and 5.52) are complex, sophisticated structures which demand experience and technical expertise, on the designer's part as well as the contractor's. As a bridge type, it offers many advantages and the development of long-span multi-stayed

Figure 5.52 Donaubrücke, Metten (courtesy of Ingenieurbüro Grassl)

decks with closely spaced cables in two longitudinal planes is very promising because of the requirement of only a relatively light and flexible deck and the inherent stability attributable to the system damping effects due to the dynamic interaction of its members.

5.8 Bridge Bearings, Movement Joints, Balustrades, Surfacing, Other Furniture and Special Details

Although bridge furniture is functionally essential and aesthetically important, it is often given belated consideration in the design of bridges. Whilst furniture such as balustrades, bearings, movement joints, signposting, lighting and drainage can constitute a relatively small proportion of the total cost of a bridge, careful consideration of details is required at the early stages of the design. The addition of some of these items at a later stage can result in substantial redesign or disfigurement of the bridge.

5.8.1 Bearings*

The function of a bearing is to transmit loads or forces in specific directions from one member of the structure to another, providing restraints against specific directional or rotational movements while allowing freedom for other movements to take place. Bridge bearings typically may be required to transmit vertical loads from the superstructure to a substructure with freedom of any required combination of movements in the horizontal plane and rotations about components of any of three orthogonal axes.

Movements of the superstructure relative to the substructures can be classified under (a) short-term, (b) long-term and permanent movements, and (c) variable or transient movements. Axial expansion or contraction and rotations due to temperature range and temperature gradient, as well as erection loads, belong to category (a). Superimposed dead loads, earth pressure, differential settlement of foundations, creep, relaxation of prestressing and shrinkage effects belong to category (b). Movements or rotations due to applied actions of traffic loading, wind forces of earthquake excitation belong to category (c).

The degree of articulation of the structure and freedom of movement at joints by means of bearings requires careful consideration in order to minimize stresses due to the aforementioned causes without unnecessarily eliminating useful redundancies or increasing the risk of instability.

* Part 9 of Ref. 5.64 covers the subject matter in detail.

The quantitative assessment of movements and forces has been dealt with in Chapter 4. It must be stressed that they should not be underestimated. The cost of enlarging the top plate of a sliding bearing initially, or of providing greater fixity, is relatively small.

5.8.2 Types of Bearing

Bridge bearings can be classified as follows:

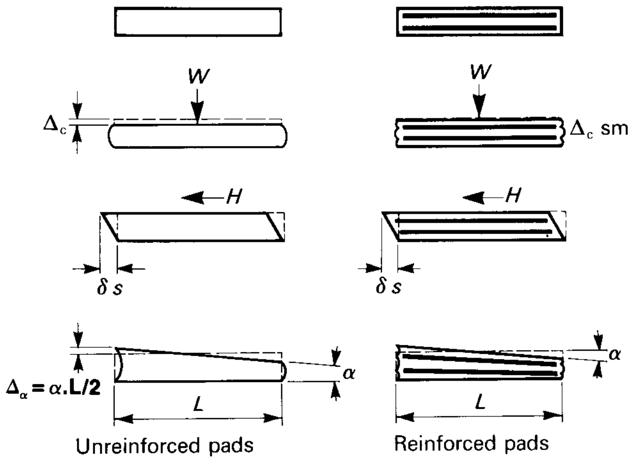
(a) elastomeric bearing pads (unreinforced or laminated), with or without sliding plates;
(b) pot bearings;
(c) spherical, cylindrical, pin or leaf knuckle bearings;
(d) concrete hinges;
(e) steel (linear or point) rocker bearings;
(f) single or multiple (cylindrical or non-cylindrical) steel roller bearings;
(g) multiple steel ball bearings;
(h) guide bearings.

Only some of the types under (a)–(d) will be described here, being those most commonly used in concrete bridges.

5.8.2.1 Elastomeric Bearing Pads

An elastomer is a substance which can be stretched at room temperature to at least twice its original length and, if released, returns to its approximate original length quickly. For bridge bearings, the term 'elastomer' applies to natural and synthetic rubbers. Bridge bearings are required to have good durability and ageing resistance; Neoprene, for example, has good resistance to sunlight and ozone. They are usually painted with a synthetic rubber paint or encased entirely in 3 mm Neoprene. Elastomeric bearings may be available ex-stock in standard sizes with given load and displacement characteristics. More usually, they are individually designed and purpose-made, often from standard sheets (see Fig. 5.53a).

Rubber, when confined, is almost incompressible, i.e. for practical purposes its volume does not alter under load; therefore for it to deflect under load it must be able to bulge laterally. High deflection under compression is generally not desirable in bridges. This is overcome by laminating thin rubber sheets between steel plates. Typical thicknesses of rubber are between 5 and 15 mm (0.2–0.6 in), whilst the steel plates are usually between 1.5 and 4 mm (0.06–0.16 in) thick. Thinner plates can be used between the intermediate layers than for the outer plates. These plates restrain the rubber and are under considerable tension,



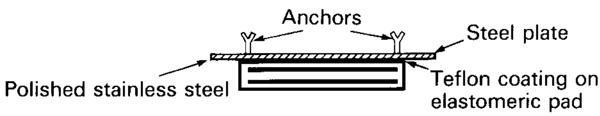


Figure 5.53 (a) Elastomeric bearing pads: W = vertical or normal load, H = horizontal or transverse load, Δ_c = compression, δ_s = shear deformation, α = angle of rotation in radians, Δ_α = extension or compression due to rotation; (b) elastomeric sliding bearing

ensuring that the lateral strain in the rubber is largely restricted to a reduced bulge at the edges. As can be seen from Fig. 5.53a, the laminations increase the vertical stiffness and the resistance to rotation. They do not, however, affect the shear stiffness. These bearings should not be subject to transverse (usually vertical) tension and compression Δ_c due to load should be greater than the maximum theoretical tensile strain due to rotation, $\alpha L/2$. Bearings subject to transverse tension are prone to delaminate. Fixed bearings should have an adequate factor of safety to resist horizontal forces. If the friction between the bearing and the sub- and super-structure is insufficient, this can be remedied by using dowels, as shown in Fig. 5.17. As bearings should be replaceable, it is recommended that these dowels do not pass through but are rather positioned alongside them, so that the required rotation is not restricted. Elastomeric bearings should preferably be placed normal to the maximum load and well bedded. If the inclination of the resultant load can vary considerably, the resultant components of shear must be taken into account. Design recommendations for rubber bearings are given in Refs 5.64–5.66. An innovation of the pure elastomeric bearing is to combine this with a sliding plate, as shown in Fig. 5.53(b).

5.8.2.2 Pot Bearings

A pot bearing ingeniously utilizes the fact that rubber is incompressible if confined. In principle, it consists of a steel pot containing a layer of natural rubber or Neoprene which supports a piston that is shaped so

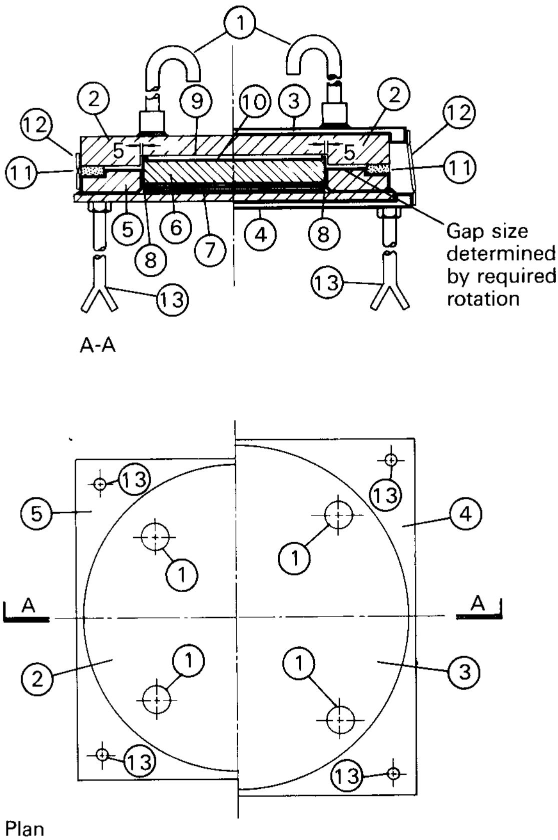


Figure 5.54 Locating pot bearing (based on Nova–Kreutz bearing): (1) upper fixing bolts; (2) top plate; (3) top adaptor plate; (4) bottom adaptor plate; (5) pot; (6) piston plate; (7) elastomer pad; (8) slip seal; (9) stainless steel disc; (10) Teflon disc; (11) pot seal; (12) failing strap; (13) lower fixing bolts with lock and locating nuts

that it can rotate about the horizontal axes. The load is applied via the piston through the rubber and pot to the substructure. Effective sealing of the pot at the perimeter of the piston face is essential as the rubber, under high pressures approaching 30 MPa (4350 psi), acts like a viscous fluid. Because of its low shear modulus, it offers little resistance to rotation of the piston, thus approximately centering the vertical load on the piston face. Translation, if required in one or more directions, is catered for by Teflon and stainless steel sliding plates. Figure 5.54 shows a locating bearing permitting 5 mm (0.21 in) movement in any direction before acting as a bearing fixed against sliding. Figure 5.55 shows a unidirectional pot bearing.

Before selecting or specifying the bearing, the bridge designer should define the following parameters:

1. Type of bearing, i.e. fixed, unidirectional or multidirectional.
2. Maximum and minimum vertical loads.

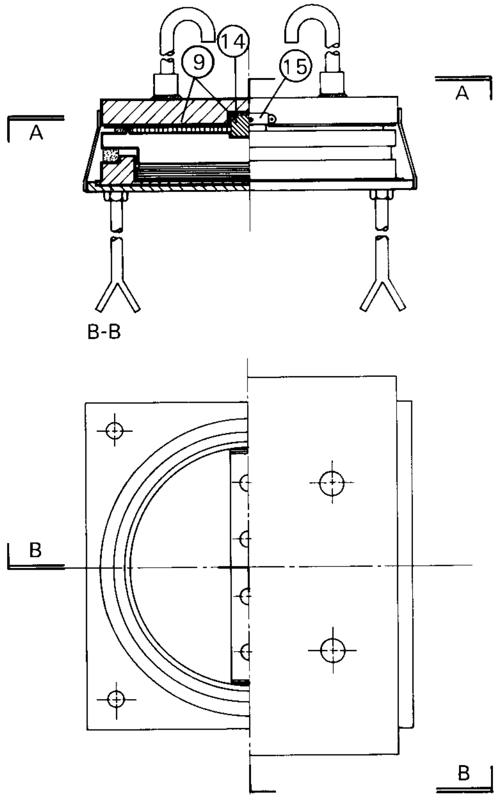


Figure 5.55 Unidirectional pot bearing (based on Nova–Kreutz bearing). Component parts as for Fig. 5.54 except the following: (9) stainless steel sliding plates; (14) keyway with Teflon sliding surface; (15) keyway wiper seal; also, there is no top adaptor plate

3. Maximum and minimum horizontal loads and their directions, if applicable. In standard bearings, the horizontal load is usually limited to 10 per cent of the maximum vertical load. Should it be required to resist a greater horizontal force, it is often more economic to design a special bearing rather than selecting a larger standard bearing.
4. Range of rotation about any axis. This must be realistically determined. Specifying too small a value may impair the function of the bearing. Too large a value increases the cost unnecessarily. The thickness of the rubber is determined by the acceptable moment due to rotation. The thicker the rubber, the smaller the moment.[5.24] Standard bearings usually allow up to 0.010 rad, but designs with up to 0.020 rad are common.
5. Movement range and preset if required. Presets less than 20 mm (0.8 in) are usually allowed for by enlarging the sliding plate. Presets must be clearly specified and marked on the bearing and the installation checked by a competent person on site.

6. Permissible concrete stresses. Many standard bearings are designed for a peak concrete stress of 20 MPa (2900 psi). Reducing this stress can significantly increase the cost of the bearings as the plates are required to be both larger and thicker.
7. The requirements for adaptor plates. These are normally an optional extra. Adaptor plates facilitate the removal of the bearing subsequent to installation, with a minimum of jacking of the superstructure. They also reduce the concrete stresses.

Criteria such as Teflon stress, elastomer stress and elastomer Shore number, are generally determined by the designer of the bearing. Common limited values are: 30–40 MPa (4350–5800 psi); 25–30 MPa (3600–4350 psi); 55–60 Shore hardness. The resistance to sliding (coefficient of friction μ) between Teflon and the sliding plate (usually polished stainless steel) decreases with increased normal pressure. Commonly accepted values are:

p/MPa	μ
10	0.055
20	0.040
30	0.030
40	0.025

(See codes and manufacturers' catalogues.) In addition, the sliding surfaces are coated with a silicon grease.

Bearings rely on uniform support conditions for satisfactory performance. Beddings of epoxies or semi-dry cement mortar rammed well home have proved satisfactory. Grouting between concrete and bearing is not recommended because of the risk of air pockets and voids being formed.

5.8.2.3 Spherical Bearings

Spherical bearings are suitable for large rotations. Permissible rotations up to 0.045 rad are standard. The resistance to rotation is higher in spherical bearings than in pot bearings. Figure 5.56 shows a Glacier fixed and unidirectional bearing. The general comments of Section 5.8.2.2 apply to this section as well.

5.8.2.4 Concrete Hinges

In principle, concrete hinges are simple. It is, however, essential that they are properly detailed and that the workmanship is of a high standard. In a concrete hinge, the section of the member is reduced to form a throat (see Fig. 5.57). In the rectangular hinge, the concrete in the throat is under biaxial stress, whereas in the circular hinge it is under triaxial stress, thus permitting high axial stress in the concrete without impairing

Figure 5.56
Spherical bearings (based on Glacier bearing)

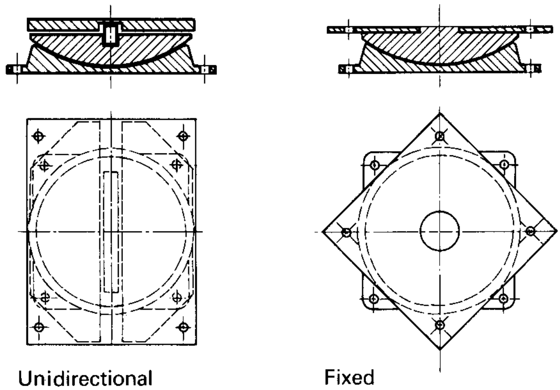


Figure 5.57
Concrete hinges: (a) rectangular hinge, rotation about *z*-axis only; (b) circular hinge, rotation about any horizontal axis

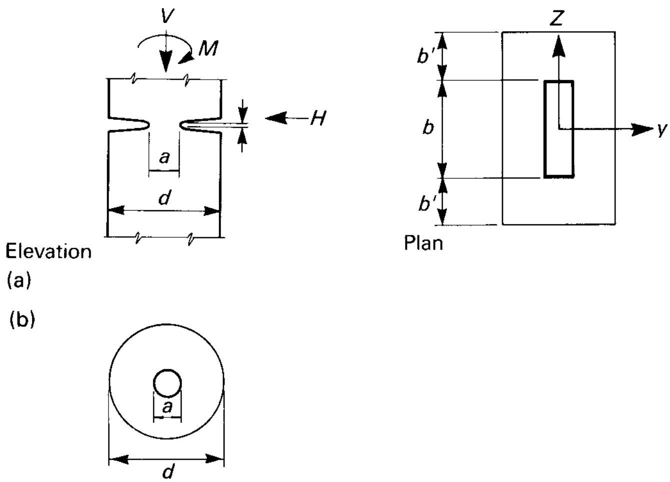
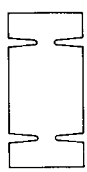


Figure 5.58
Concrete rocker bearing



the safety of the hinge. The UK Department of Transport allows working stresses up to twice the cube stress. The small dimensions of the throat permit rotation to take place with little restraint. It is not necessary to provide vertical reinforcement across the hinge unless there are exceptional circumstances. If reinforcement is provided through the hinge, it should consist of small-diameter bars located in the line of bending in the centre of the throat. These increase the resistance to rotation. Small rotations are accommodated by increased concrete strain. Large rotations cause the concrete to crack. Tests have shown that this does not impair the functioning of the hinge, even with reversals of rotation causing the concrete to crack on either side of the hinge. The corners of the throat should be well rounded. A parabolic shape is considered ideal to prevent the concrete from spalling off. For this reason, it is also advisable to constrict the throat in the transverse direction in the rectangular hinge. The resistance to transverse shear is limited and is a function of the axial compression. It can be enhanced by the use of dowels. It is essential that the throat is under residual compression under all load cases, if necessary by means of prestressing across the axis of rotation, if possible outside the throat area. The member of each side of the throat is subject to high splitting forces which must be adequately catered for by suitable reinforcement, similarly to a prestressed concrete endblock. Detailed design procedures for concrete hinges are given in Refs 5.42 and 5.67. Concrete hinges can also be used in rocker bearings, thus allowing translation as well as rotation (see Fig. 5.58). As the restricted throat area and congestion of reinforcement make concreting difficult, concrete hinges are often precast.

5.8.2.5 Other Bearings

The bearings as listed under (e)–(h) in Section 5.8.2 are more often used in steel bridges, but may be applicable to large concrete bridges. Certain bearings, such as the pot and spherical bearings, can be adapted to alter their function during their lifetime, e.g. a temporarily fixed bearing can be altered to be a permanently uni- or multi-directional bearing or vice versa (useful for stage construction). The reader is referred to manufacturers' literature on the installation and use of these bearings. Another type of bearing is that used for incremental launching described in Section 5.3.

5.8.3 Movement Joints

The main functional requirements of joints are as follows:

1. To bridge the variable gap due to expansion or contraction in the carriageway in such a manner that under all circumstances no objectionable discontinuities occur in the riding surface of the carriageway. The riding quality should not cause inconvenience to any class of road user for which the structure is designed. Where applicable, this includes pedestrians, cyclists and animals.
2. To withstand the loads due to traffic, vibration, impact, shrinkage, creep, settlement, etc., without causing excessive stresses in the joint or elsewhere in the structure.

3. Not to constitute a traffic hazard, e.g. skidding.
4. Not to give rise to excessive noise or vibration.
5. Either to be sealed against the ingress of water and debris, or to have provision for drainage and cleaning.
6. To be easy to inspect and maintain.
7. To be economic in their total cost, including initial capital outlay and maintenance.

It is probably safe to say that no other single component of a bridge has caused as many service problems as expansion joints. Low-cost joints have often proved to be expensive in maintenance and replacement costs, quite apart from the inconvenience and disruption of traffic.

5.8.3.1 Suitability of Joint Types for Movement Ranges

There are two types of gap: covered and open. In the context used here, a covered gap is one which is structurally bridged such that loads are carried across the gap. It does not relate to the water resistance of the joint. An open gap is one which is not structurally bridged; for example, a gap filled with compressed bitumen-impregnated high-density polyurethane foam might be watertight, but the foam cannot transmit loads — therefore the gap is an open gap.

It has been stated that for joint movements:[5.67,5.68]

(a) if the gap is less than 10 mm (0.4 in) the surfacing can be taken across the joint but a bond breaker should be applied to limit the crack widths in the surfacing; for gaps of less than 5 mm, no special precautions need be taken;
(b) the maximum open gap acceptable for vehicles is 65 mm ($2\frac{1}{2}$ in);
(c) covered gap joints should be provided for pedestrian, animal and bicycle traffic;
(d) open gap joints should have a minimum gap of 6 mm ($\frac{1}{4}$ in).

Small movements, i.e. 10–40 mm (0.4–1.6 in), are economically catered for by means of flexible sealers such as polysulphide, or inserts such as precompressed impregnated foam or pre-formed Neoprene strips. Joints for larger movements are usually specially designed. A range of proprietary types is available. These are usually made of steel in the form of combs with interlocking teeth, articulated sliding plates that move in under an overlapping end plate, or various versions of an assembly of transversely positioned rolled steel or extruded aluminium channels, I-sections or plates with compressible hollow or folding rubber infill sections between them, but these will not be described herein. The technical literature provided by the suppliers should be consulted.

The two-part polysulphide sealant is suitable for movements in

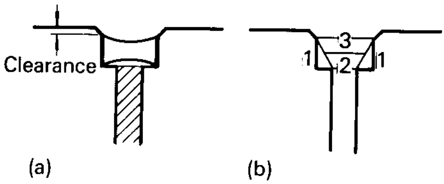


Figure 5.59 Polysulphide joint: (a) sealant under tension; (b) placing sequence for wide sealant

compression and tension of 25 per cent of the sealant width is installed. The permissible shear movement is higher, namely ± 50 per cent of the joint width as installed (see Fig. 5.59a).

In horizontal joints, the limiting vertical movement is therefore usually the maximum acceptable step between the two sides of the joint. Where the movement is predominantly horizontal, i.e. across the joint, the width-to-depth ratio should be 2:1 (minimum depth 10 mm). The maximum width of a joint which can be installed in one application is about 25 mm (1 in). Greater widths can be sealed by applying the compound in stages, allowing each application to set before applying the next (Fig. 5.59b).

The surface preparation of a joint prior to installation of the compound is vital. Surfaces must be sound, clean and dry. The back-up material in the joint is important; it regulates the depth of sealant and serves as a bond breaker allowing free movement of the sealant. Surfaces must be primed and the joint installed strictly in accordance with the manufacturer's recommendations. An important point is to take cognisance of the suction action of tyres travelling at speed across the joint. The joint must be adequately recessed so as not to be whipped out by traffic. A clearance of 15–20 mm (0.6–0.8 in) is recommended. If the top of the joint is too deep below the surface, it will fail as a result of debris. Polysulphide itself is not trafficable. Its use in carriageways has been limited. It has, however, been extensively and successfully used in sealing kerbs and balustrades.

The movement range of bitumen-impregnated high-density polyurethane foam depends on the functional requirements of the joint. It has no tensile resistance and for satisfactory function depends on remaining under adequate compression. Table 5.1 shows the claimed characteristics with

Table 5.1

Compression/%	Effect
50	Dust seal
$33\frac{1}{3}$	Draught seal
25	Water resistant
20	Watertight
15	Hydrostatic and vacuum seal
10	Liable to compression set

respect to compression. The recommended vertical shear movement is ± 10 per cent of the seal depth. In road carriageway crossings, the minimum recommended depth of seal is 75 mm (3 in). The material should not be used where there is the likelihood of dirt or debris being forced into it. Petroleum products dissolve the bituminous impregnation. Ultraviolet light degrades the upper surface. The depth of degradation is generally small so that the joint function is not impaired. Taking cognisance of the maximum acceptable 'open gap' joint width of 65 mm ($2\frac{1}{2}$ in) and desirable water resistance, the total movement range is 30 mm (1.2 in). The top of the joint should be recessed 20 mm (0.8 in) below the roadway surface.

Compression seals are available in sizes to cater for movements in excess of 80 mm. However, with a maximum open gap of 65 mm, the movement range is approximately 30 mm. The seals are dependent on a minimum compression for satisfactory performance, the minimum compression normally being 10 per cent and the maximum 55 per cent of the uncompressed width. The depth-to-width ratio of the uncompressed seal should be at least 1:1. As above, depth of installation and surface preparation are critical. Also, the correct type of seal should be chosen. For bridge joints, the edges of the seal should be positively seated (see Fig. 5.60). The seals cater for a shear movement of an angular rotation of up to 10 per cent.

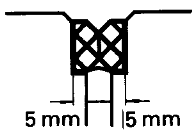


Figure 5.60
Neoprene compression seal

5.8.3.2 Nosings

The above-mentioned joints are generally protected by nosings. These are usually of concrete, epoxy or polyester mortar, or steel. These nosings receive a severe pounding under traffic and should be robust enough to withstand these forces.[5.24,5.67,5.68] Their tops should be level with the top of the wearing course to minimize impact loading. To ensure smoothness of ride, it is common practice to lay the wearing course first, then cut out and construct the nosing. For concrete and epoxy/polyester nosings, the surface preparation is vital to ensure adequate bonding. Transverse joints should be supplied at sufficient intervals — 900 mm (3 ft) for epoxy nosings — to cope with differential shrinkage and temperature. Most epoxies are moisture-sensitive. Polyesters are exothermic. The manufacturers' recommendations should be strictly adhered to. Correctly used, epoxy is an excellent material, yet failure rates of epoxy nosings have been extremely high, mainly because of faulty workmanship.

5.8.4 Balustrades, Handrails and Other Details

5.8.4.1 Function

The main function of balustrades or handrails is to prevent traffic and pedestrians respectively from crashing and falling over the edge. Another function of balustrades is to minimize the consequences of an accident. Vehicles should not overturn or be thrown into the face of oncoming traffic. Opinions as to the correct solution differ. Figure 5.61 shows two concrete balustrades. In type (a), the flexible approach guardrails extend for the length of the bridge. Type (b) is shaped so that when a vehicle strikes the concrete balustrade at an obique angle, the wheel travels up the lower slope and is then deflected back and downwards. The maintenance costs of this type of balustrade are low and damage to the vehicle and its occupants usually less than for flexible types that function on the principle of absorbing energy by yield deformation of the components. Leonhardt[5.24] has proposed cables strung to spring-loaded posts with shear notches at the bottom so that during an impact the posts will shear and not be flattened, the cables thereby restraining the vehicle. All types of balustrade should be designed for the specified factored impact loads, but should fail prior to the load-carrying component of the deck to which they are attached.

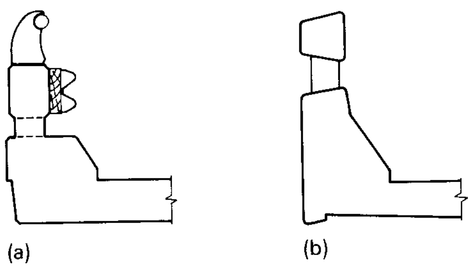


Figure 5.61 Concrete balustrade sections: (a) approach guard rails continue over bridge; (b) approach guard rails end at bridge

Balustrades are visually a dominating part of the bridge (Fig. 5.62). They should therefore not only be functional, but aesthetically pleasing. For long-span structures such as suspension bridges, the wind effects on the balustrades are significant so that the projected area and shape should be optimized.

5.8.4.2 Kerbs

The kerb configuration is usually specified by the road authority. In cases where the deflection of the deck may vary during construction, it is recommended that kerbs be constructed only after the deck structure is sufficiently advanced.

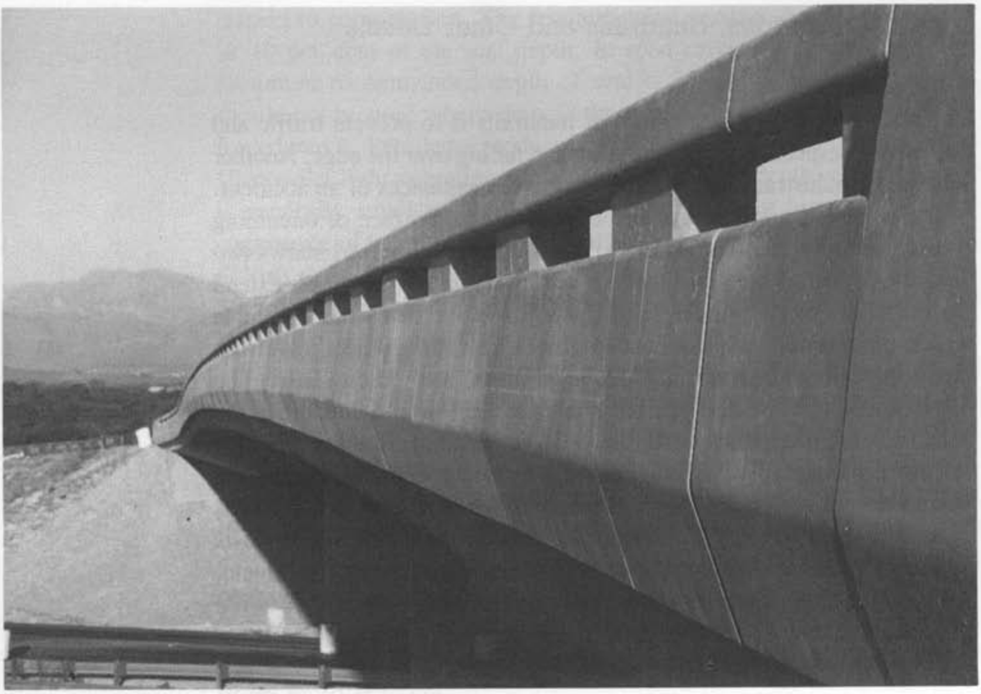


Figure 5.62 Concrete balustrade

5.8.4.3 Wind Deflectors

On high bridges exposed to strong winds, balustrades may be fitted with wind deflectors.

5.8.4.4 Noise Abaters

The noise level generated by traffic on main roads may reach intolerable proportions. Large sections of freeways in heavily built-up areas in Japan, for instance, have been provided with noise deflectors. These might alleviate the noise problem for riparian owners, but they are aesthetically displeasing and create a tunnel effect for the motorist. It still appears that, from an aesthetic and noise pollution point of view, elevated motorways in heavily built-up areas are unacceptable to many. Sunken freeways may offer a better solution but are costly. This is an aspect which requires the attention of town planners.

5.8.5 Surfacing

The most common surfacing for bridge decks is bituminous premix 40–75 mm (1.6–3 in) thick. In countries subject to frost, especially when de-icing salts are used, effective waterproofing to the deck is essential.[5.24,5.69] The consequences of corrosion of steel due to penetration of water with a high chloride content where de-icing salts were applied to the deck coupled with damage to concrete due to freeze–thaw action and the bursting effects of the corroded steel (see Chapter 6, Section 6.2) has taken on alarming proportions in many cases where inadequate protection procedures were implemented. Membranes have not always been successful but various improved coatings are being developed. In areas not subject to frost, the common procedure is to clean the concrete surface by washing and brushing, to allow the surface to dry and then to cover it uniformly with a spray-grade bituminous emulsion. After the tack coat has dried, the premix is applied.

5.8.6 Drainage

Drainage details should be considered at an early design stage. The frequency of outlets is dependent on the intensity and duration of rainfall, the slope of the surface and the slope and size of drainage pipes. To avoid standing water, a minimum deck slope of 2 per cent is recommended. This is not possible at changes in superelevation. These areas therefore require special consideration. For bridges in open countryside, the water can be drained directly through scupper outlets in the deck.

The inlet ends of these scuppers should be detailed so that they do not present a hazard to animals or pedestrians. In built-up areas, water will generally have to be led to stormwater systems. Pipes cast into concrete should be 150 mm (6 in) in diameter unless they are readily accessible for cleaning, which is unusual. Exposed pipework should be avoided as this can spoil the appearance of a bridge. If drainage pipes cannot be accommodated in the concrete, they should be placed in recesses under the deck. All bends should be provided with rodding eyes. In areas subject to frost, drainage pipes should be designed so that, if they freeze up, they do not cause structural damage. Premix itself is not waterproof. The water in the premix should also be drained to avoid pumping and failure of the premix.

5.8.7 Services

The requirements of signposting, street lighting and the need to carry services such as electric cables, telephone cables, water pipes, etc., across the bridge, should be established early on and adequate provision for their support and accommodation made. It is often technically and aesthetically extremely difficult to make provision for these services when the design is well advanced or construction has commenced.

5.8.8 Detailing of Structural Elements

The detailing of reinforcement can have a marked influence on the performance and reliability of a bridge structure (see Chapter 13, by Taylor, in Ref. 5.1). This applies particularly to mechanisms of force transfer by bond and bearing at anchorages, splices and corner joints as well as to bursting forces in curved members and at anchorages of prestressed beams. Checks on bond stresses are imperative where the rates of change of forces in reinforcement can be high, e.g. high local bond stresses on reinforcement due to large shear forces acting in conjunction with bending forces. Where the bars extend beyond the section of high stress, such localized overstress does not necessarily imply imminent failure, provided auxiliary mechanisms of resistance are available. Where simplified methods of design are used in which internal forces are resisted by equivalent systems of concrete struts and steel ties, such as that developed by Ritter and Mörsch and used by many designers for several decades but more recently revived by various authors such as Schlaich,[5.70] in a more systematic and generalized manner, care should be taken in detailing the reinforcement so that a reasonable structural system of resistance can be shown to exist within the body of the concrete member. Such 'truss models' are assumed to consist of discrete concrete compressive members interacting with reinforcing steel tension and compression members that are in stable equilibrium under load with approximate strain compatibility and adequately connected without exceeding permissible stresses. Structural steel plates or rolled steel sections are often incorporated as anchors or to distribute high stress concentrations.

These methods of analysis are useful as approximately lower-bound solutions for the design of deep beams and corbels or half-joints and nibs, and are the basis of some of the formulae developed by space truss analogy theory for the design of shear reinforcement in beams and torsion in box-beams. The non-elastic or semi-plastic behaviour of concrete in conjunction with the yield properties of mild steel make reinforced concrete a very 'forgiving' material. These methods, where they have been related to empirical results or more refined analyses using finite elements, are usually adequate for normal design.

When large structural members are involved, however, care should be taken that permissible strains are not exceeded prior to attaining the theoretical ultimate limit states, especially where the analysis is based on the assumption that plane sections remain plane under bending action. A typical example would be the necessary requirement to limit the permissible plastic rotation of continuous deep beams in ultimate limit state analysis.

Where large-diameter bars in compression are lapped, very high local stresses may occur where the ends of the bars butt against concrete, and have been shown in tests to cause premature failure due to splitting or spalling of concrete in cases where normal concrete cover has been provided. In such cases additional cover and steel binders should be provided. About 30 years ago the author was faced with this situation, created by the architect severely reducing certain column sizes for a 20-storey building. The problem was solved by providing soft rubber caps to the ends of the bars. The columns have shown no signs of distress.

For zones in the concrete structure where inelastic behaviour has been assumed likely as a result of earthquake effects and where reversed cycles of deformation may be repetitive, sufficient transverse confinement reinforcement should be provided to prevent splitting or tension cracks, thereby ensuring improved ductile performance and increased strength and strain capacity.

An important requirement that is often overlooked by inexperienced detailers is that of allowing enough space and access for the placing and vibrating of the concrete (Fig. 5.63).

The importance of adequate concrete cover and the need to ensure a dense concrete, especially at and near exposed surfaces, cannot be over-emphasized. Codes of practice usually specify minimum requirements for various environments (see also Section 5.8.5 and Chapter 6, Section 6.2).

5.9 Estimation and Optimization of Costs by Value Engineering

In previous sections, various values related to the design of bridges have been discussed including the importance of reliability and utility together with aesthetic excellence and the importance of environmental aspects. The total cost, which by implication is a very important part of the utility function (see Chapter 3, Section 3.2), needs full consideration from the very first stages of the conceptual design.

At the initial stages, cost valuations are done by estimating. The assumptions made and method used will depend on the stage of development of the design. Accurate estimating is difficult unless it is based on considerable experience of similar work. Cost graphs can be prepared for total structures or parts of structures in terms of specific variables and concomitant parameters. Interpolation can give useful results if the cost graphs are based on experience covering a wider range of structures than the one under consideration. Extrapolation is often necessary but needs great care. Unknown aspects of new projects should best be analysed in fair detail although this may be very difficult if complex operations are involved. It is reasonably easy to estimate material costs, but plant and man-hour content as well as overheads are often

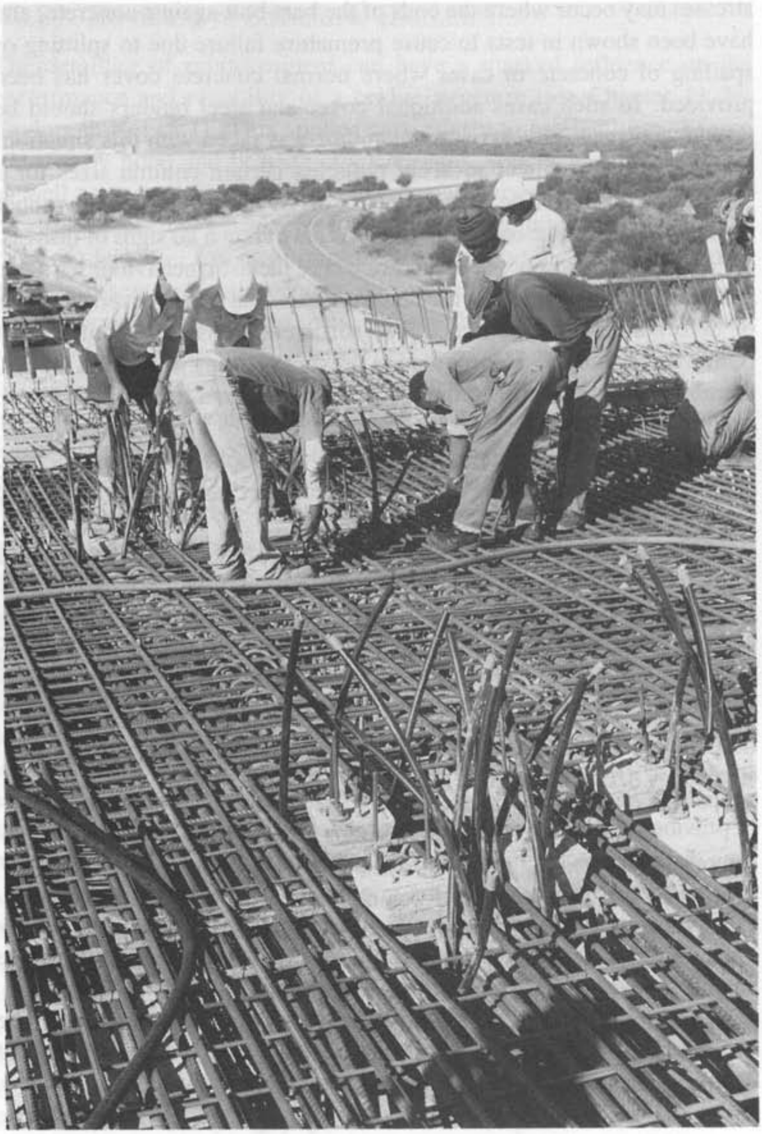


Figure 5.63
Reinforcing steel in bridge deck

difficult to quantify without previous experience. Even with previous experience, the application of cost extrapolation to larger or more complex works can be very inaccurate.

It is consequently necessary to update estimates regularly as the design proceeds and the details become better defined. When the design is fully developed and a more complete schedule of quantities can be prepared, it remains to determine more reliable cost rates. Estimates should always be done at present costs and the projected additional costs due to escalation indicated separately to avoid misunderstanding as to the probable costs at time of tendering. If the contract documents make provision for contract price adjustment formulae to allow for escalation due to inflation or the opposite, then the employer should be advised on the probable end cost as well.

During the total period of design and construction, it should be the objective to optimize costs in terms of utility, as defined in Chapter 3, Section 3.2. This is naturally part of the design process as depicted in Fig. 3.1 and requires knowledge, experience, proficiency and the ability to innovate.

The concept of value engineering[5.71] or value analysis began during the Second World War when shortages of materials and labour necessitated changes in method, materials and traditional designs. Many of these changes resulted in superior performance at a lower cost.

Value engineering or analysis as defined by Miles[5.72] is 'an organized, creative approach which has for its purpose the effective identification of unnecessary costs, i.e. costs which provide neither quality nor use nor life nor appearance nor customer features'.

Although the techniques inherent in this process have in the past largely been applied to multidisciplinary works, it applies equally to civil engineering. It is applicable to the total process from conceptual design to construction of bridges.

The considerable literature on the subject should be studied by designers and constructors.

5.10 Contractual Procedures

Contractual procedures differ from country to country and according to the requirements of a particular project. The following are definitions of the types of contract based on British practice:

(a) A *fixed price contract* is one wherein a price is agreed and fixed before construction starts. It may or may not be a firm price contract. Usually, it has priced bills of quantities for basis.
(b) A *firm price contract* is a fixed price contract which does not allow for its prices to be adjusted for fluctuations in normal market prices of labour and materials.
(c) A *lump sum contract* is a fixed price contract wherein the contractor agrees to be responsible for executing the whole of the contract work for a stated total sum. This type of contract would normally be chosen only when few variations are expected. Usually, it is based upon drawings and specification.
(d) A *measurement contract* is a fixed price contract wherein prices are fixed in advance for units of work to be measured later. These are

recorded in a schedule of prices. The total work to be done is not usually decided in full detail before the contract so the total price cannot be ascertained until the work is measured, unless a provisional bill of approximate quantities is produced.

(e) A *cost-reimbursement contract* is one wherein the actual prime costs of labour, material, subcontracts, use of plant, etc., will be paid for at net cost to the contractor plus a fee. The fee is intended to pay the contractor for his management costs, overheads and profit. The fee may be a sum wholly or partly fixed in advance or it may be a percentage of the prime cost. The contract should define what is to be deemed a prime cost and what is to be deemed as being paid for in the fee.

(f) A *target price contract* is one where the contractor undertakes not to exceed an agreed ceiling figure. The financial basis of the contract is somewhat similar to a cost-reimbursement contract with a sharing (between client and contractor) of any savings achieved below the target figure.

(g) A *negotiated contract* is one which is negotiated between the client or his agents and a selected contractor. The price basis may be any of the foregoing.

(h) A *managed contract* is a form of negotiated contract wherein the contractor plays a more positive role in the time/cost control of the contract in the client's interest.

(i) A *packaged contract* is similar to a managed contract but the contractor undertakes all responsibility for the design and eventual performance of the bridge.

The measurement contract with a provisional bill of approximate quantities is commonly used by government authorities. Although many authorities have a bridge design capability, it is common practice to appoint consultants for the design of selected bridges. The consultants' major duties are normally the following:

(a) To prepare a preliminary design(s), cost estimates and reports including, where applicable, feasibility, comparative and environmental impact studies based on sufficient investigations, surveys and analyses.

(b) On acceptance of (a), to proceed with detailed design and preparation of contract documents. These usually consist of conditions of contract, such as those prepared by the ICE[5.73] or by the FIDIC,[5.74] specifications, plans, details, and schedules of quantities.

(c) To call for tenders either by open tender (advertised) or by selective tendering, where each prospective tenderer is invited. Alternative designs may or may not be considered. This is stated in the invitation to tender.

(d) To evaluate tenders, which may include some or all of the following: to evaluate the rates, to assess the implications of qualifications of tender, to report on tenderers' technical competence and resources, their ability to comply with the completion date and their financial standing. At times of high inflation and high interest rates, cash flow studies in terms of the tenderers' construction programmes are usually required to compare the effective tender prices.

(e) To advise on and arrange the award of the tender and the signing of the agreement between the employer and the contractor.

(f) To supervise construction. Usually, resident engineers are appointed for full-time supervision at the site in order to ensure the quality of materials and workmanship. They should be fully briefed on their duties. Regular site meetings with representatives of the client and the contractor, and technical meetings with representatives of the contractor and subcontractors, should be held and minutes kept thereof. Full records of all observations, recordings and measurements are necessary. A daily logbook should be kept, including recordings of inclement weather and other factors that may lead to claims for extension of the completion date and/or additional costs.

(g) To check the measurement of quantities and control the valuation of claims and the preparation of payment certificates.[5.75]

There are variations to the above; for instance, tenders may be called after stage (a), without a detailed design, in which case the design parameters and requirements, any restrictions — e.g. openings for shipping, preference for large spans — are then clearly set out and tenderers are invited to submit tenders on the preliminary design proposal(s), or on alternative designs. In this way, a particular contractor's specialized knowledge or construction equipment may be of advantage. This procedure requires adapted contract documents, depending also on whether the employer commissions a consultant or not. The allocation of responsibility for design, the preparation and approval of detailed drawings, the checking, quality control and quality assurance of the construction and the procedures for measurement and payment, are matters which require clarification therein.

Quality control of construction materials and workmanship can be defined as the regulatory process through which we measure actual quality performance, compare it with standards, and act on the difference, e.g. controlling the quality of concrete by sampling, preparing cubes or cylinders, and testing. If the strength is below that permissible the concrete is either rejected or permissible loads on the structure are reduced accordingly. Quality assurance of design and construction procedures that require stringent application can be defined as the activity of providing, to all concerned, the evidence needed to establish confidence that the quality function is being performed adequately.[5.76]

An extreme version of contract, namely, the 'packaged contract', places full responsibility for design and construction, including supervision and decisions on compliance with standards of acceptance, on the contractor himself, who is then fully responsible for the eventual performance of the bridge.

There are strong differences of opinion about the merits of these methods, largely because of the subjective element in decisions relating to quality and standards and the natural tendency for the contractor in a competitive situation to minimize costs within the terms of the contract. Theoretically, the latter procedure may result in an increase of risk as defined in Chapter 3, Section 3.2.

It is beyond the scope of this handbook to enter into a detailed analysis of the pros and cons of the various national and international contractual procedures. The reader is referred to Refs 5.77 and 5.78 for comprehensive studies in this respect.

It is also very important that responsible engineers acquire some understanding of legal liability under the law of the land in order to act correctly in contractual matters and to take the necessary precautionary steps, whether in design, contract supervision or construction. It is beyond the scope of this book to deal with such matters in any detail but, beyond becoming well acquainted with all relevant contract documents, the engineer involved in any work should for detailed knowledge of the law consult more experienced engineers or lawyers when in doubt. He should in every case take due care to ensure, in the light of the well-known common law test for negligence, that his actions and standards of work never fall below those which could be expected of a reasonable and competent engineer under similar circumstances.

5.11 The Monitoring of Bridge Behaviour

Under this heading we do not understand the routine type of control involving field inspection for defects as referred to in Chapter 6, but rather the monitoring of bridge behaviour to gain information about its performance when subjected to various actions for comparison with the results of theoretical analyses and laboratory experiments. Such studies are also useful, however, in providing data for the updating of codes of practice and the rating of bridges with defects that have been identified by the above-mentioned field inspections. The objective in such cases would be to prove and quantify the effect (if any) of such defects as well as the overall effect of various redundancies that are usually ignored in the analysis and design (especially in the case of slabs; see Section 5.5.2.1) but which enhance the load-bearing capacity of the structure.

Furthermore, the various partial safety factors used in the design allow for the probability of defects that might not exist in the structure (see Chapter 3, Section 3.2) and which such observations may prove. These studies are particularly useful for the serviceability rating of bridges subject to special permit loadings where such loadings may result in excessive deformations and/or overstressing of members but with an acceptable ultimate load factor. It may be necessary in such cases to carry out observations at regular intervals over an extended period of time in order to establish whether there are signs of crack development or progressively increasing non-elastic deformations due to cyclic effects with the risk of eventual fatigue failure.

Whereas concrete bridges designed in compliance with codes of practice do not, under normal loading, often exhibit fatigue failure, the regular overloading beyond permitted service loads can cause cracking and deterioration of concrete and fatigue failure of steel reinforcement.

The necessary studies can best be done by selecting particular bridges that are for one or other reason prone to fatigue or malfunction. This would involve the instrumentation of the bridge in order to record deflections and strains with portable strain transducers at selected positions, and the measurement of traffic loadings by methods such as weigh-in-motion systems. Failing such sophisticated methods, it is extremely difficult to obtain reliable information about load spectra. However, visual observations or photographic records of traffic can be used as basic data to generate random traffic by computer.[5.79] In this manner, probable traffic load spectra can be determined and used in conjunction with other observations in the field to assess the expected useful future life of a bridge (see Chapter 6, Section 6.5). Concrete strain readings, although useful to indicate trends in behaviour, are not always quantitatively reliable for assessing stresses because of micro-cracking and the unknown prior stress–strain histories, which make the accurate assumption of the constitutive relations of the concrete impossible.

The passage of very heavy abnormal loads, especially those transported on multiwheeled trailer combinations with controlled hydraulic suspension, can present a very useful opportunity. By previous arrangement with the conveyors and the authorities, the movement over the structure can be controlled to gain useful information about deflection and sway behaviour of the structure for comparison with theoretically predicted values.

The systematic and regular recording of bridge deformation profiles to study effects that are time-related, such as creep, shrinkage and foundation settlements, is useful in checking the validity of the theories that were applied in the design or for the derivation of improved formulations. During construction of a bridge, datum targets should always be inserted into the columns and then levelled. This will then form a useful basis for the evaluation of any future foundation movement. By comparison with most laboratory experiments involving creep and shrinkage effects, such monitoring in the field is especially useful because of the elimination of the scale effects with respect to volume-to-surface ratios. The structure is also in an actual environment which is not readily simulated in a laboratory.

In the case of long-span slender concrete cable-stayed bridges with tall towers or pylons, the monitoring of the effects of wind action (see Chapter 2, Section 2.4.4) could serve a useful purpose in assessing theoretical procedures. This applies particularly to the vibration of cables and the probability of fatigue effects.

River-bed scour due to the rapid and turbulent flow of rivers in flood is probably the most common cause of bridge failure (see Chapter 2, Section 2.4.5). It is very difficult to predict the exact nature or extent of scour action, even by the use of scale models. This has made it imperative to devise some methods of monitoring scour action round piers and abutments of bridges where the failures are mainly caused by the undermining of the foundations. The visual inspection by divers or the sounding of the river-bed surface profiles subsequent to flooding does not necessarily give a true indication of the effective depths of the scour action as sand may be disturbed and redeposited at a reduced density for a considerable depth below river-bed level. Dutch cone penetrometer tests may be useful in determining relative densities after floods, from which it may be possible to deduce approximate scour depths. A simple monitoring system has been devised at the UK's Hydraulics Research Unit which is claimed to be able to give early warning of river-bed scouring. Their system uses a series of sensors which are buried in the river-bed sediment, each one is attached to a rugged steel conduit fixed firmly to the pier or abutment and connected by cable to a monitoring indicator visible from the bridge deck. When flow is normal, with no scouring, the sensors remain buried and still. As the river rises and scouring increases, the water drags away the sediment, exposing the sensors to the flow. This causes them to oscillate and trigger a surface alarm. With a series of sensors at different levels, an indication of the depth of scouring can be obtained.

In regions where the risk of earthquakes is high and where there is a relatively high recurrence of minor tremors, useful deductions may be made by monitoring the behaviour of potentially critical bridge components such as piers, abutments and joints with foundations and deck members. Such observations will not however necessarily identify all the potential critical modes under more severe earthquakes because of the complexity and non-linearity of behaviour (see Chapter 2, Section 2.4.6).

Tests performed according to the terms of the contract, on structures or parts of structures that have failed to comply with the specifications for quality of materials and/or workmanship, may provide useful opportunities for verifying design assumptions and analytical procedures.

Where failures or partial failures have occurred and repairs are carried out, the subsequent proving tests may have fundamental value, especially if the type of structure or member is novel or of a complexity that needs confirmation of the theoretical analyses.

A problem may arise in performing proving tests that require an applied loading in excess of the unfactored design loads. Such loads can cause cracks that may otherwise not occur in the lifetime of the bridge. This needs careful consideration (see Chapter 6, Section 6.5.3). Where such cracks are of no serious significance the full test load should be carried out. Where the 'damage' may be significant, a reduction in the degree of overload may be justified, provided that the probability of a lifetime loading in excess thereof is very low.

References

5.1 Kong F K, Evans R H, Cohen E, Roll F (eds) 1983 *Handbook of Structural Concrete* Pitman, London

5.2 Pennels E 1978 *Concrete Bridge Designer's Manual* Cement and Concrete Association, London, 164 pp

5.3 Hambly E C 1970 *Bridge Deck Behaviour* Chapman and Hall, London, pp 187–203

5.4 Lin T Y, Burns N H 1981 *Design of Prestressed Concrete Structures* 3rd edn, Wiley, New York, 752 pp

5.5 Neville A M 1970 *Creep of Concrete: Plain, Reinforced and Prestressed* North Holland, Amsterdam, 622 pp

5.6 Rowe R E 1962 *Concrete Bridge Design* Wiley, New York, 336 pp

5.7 Leonhardt F 1979 *Vorlesungen über Massivbau* vols 1–6, Springer Verlag, Berlin

5.8 Leonhardt F 1964 *Prestressed Concrete: Design and Construction* 2nd edn, Wilhelm Ernst & Sohn, Berlin, 677 pp

5.9 Lee D J, Richmond C J 1975 Bridges. In Blake L S (ed) *Engineer's Reference Book* 3rd edn, Newnes–Butterworth, London, pp 18.2–18.71

5.10 *Symposium of Concrete Bridge Design* SP-26 General Chairman of Symposium: Shu-t'ien Li vols I and II, American Concrete Institute, Detroit, 1971

5.11 Wittfoht H 1972 *Triumph der Spannweiten* Beton Verlag, Düsseldorf, 314 pp

5.12 Büchting F 1976 Brücke über den Europakanal aus fertigteilen mit Verbindungsfungen aus Epoxidharz *Bauingenieur* **51**: 136–43

5.13 Schlub P 1981 Formwork launching girders *IABSE Periodica* **4**; *IABSE Surveys* **5–18**: 45–68

5.14 Wittfoht W *et al* 1981 *Bridges and Formwork: Launching Girders, IABSE Periodica* **2**: *IABSE Structures* C-17, 29–43

5.15 Langraf Th. 1978 Lehrgerüstarbeiten für den Mittelteil der Hochstrasse Sündumgebung Oldenburg — Bauwerk OL-53 *Bauingenieur* **53**: 349–54
5.16 Wittfoht H 1974 VII Internationaler Spannbetonkongress New York Brücken-Herstellungsfahren (Konstruktion) und bermerkenswerte Bauwerke Bericht *Beton- und Stahlbetonbau* **70**: 16–24
5.17 Muller J 1975 Ten years of experience in precast segmental construction *J. Prestressed Concr. Inst.* **20** (1): 28–61
5.18 Schambeck H 1976 Brücken aus Spannbeton: Wirklichkeiten, Möglichkeiten *Bauingenieur* **51**: 285–98
5.19 Wittfoht H 1970 Die Siegtalbrücke Eiserfeld im Zuge der Autobahn Dortmund–Giessen *Beton- und Stahlbetonbau* **65**: 1–10
5.20 Sommerer R, Wösener K, Nehse H 1978 Die Eschachtalbrücke im Zuge der BAB Stuttgart–Westlicher Bodensee *Bauingenieur* **53**: 117–26
5.21 Buchting F, Moosbrugger P 1976 Brücke über den Europakanal aus fertigteilen mit Verbindungsfugen aus Epoxidharz *Bauingenieur* **51**: 137–43
5.22 Guckenberger K 1977 Neuere Entwicklungen im Spannbetonbau mit Kunstharz verklebten Fertigteilen *Bauingenieur* **52**: 456–8
5.23 Nelson Pedersen H, Muller J, Wittfoht H 1977 Vom Bau der Sallingsund Brücke *Beton- und Stahlbetonbau* **72**: 77–85
5.24 Leonhardt F 1979 *Vorlesungen über Massivbau* vol 6: Grundlagen des Massivbrückenbaus Springer Verlag, Berlin
5.25 Barker J July–Aug 1980 Construction techniques for segmental concrete bridges *J. Prestressed Concr. Inst.* **25** (4): 66–86
5.26 Leonhardt F, Baur W 1971 Erfahrungen mit dem Taktschiebe-verfahren im Brücken und Hochbau *Beton- und Stahlbetonbau* **66**: 161–67
5.27 Bonatz P, Raquet V, Seifried G 1979 Brücken Trichtinbachtal und Ramersdorf — zwei neuartige Anwendungen des Taktschiebe-verfahrens *Beton- und Stahlbetonbau* **74**: pp 29–35
5.28 Stumpp A 1975 Herstellung einer Unterführung unter Bundesbahngleisen nach dem hydraulischen Vorpressverfahren *Beton- und Stahlbetonbau* **70**: 113–15
5.29 Bareš R 1968 *Tables for the Analysis of Plates, Slabs and Diaphragms Based on the Elastic Theory* Bauverlag, Wiesbaden
5.30 Stein D 1966 Brückenflügel als an zwei benachbarten Rändern eingespannte Platte *Beton- und Stahlbetonbau* **3**: 62–8
5.31 US Department of Transportation 1970 *Hydraulics of Bridge Waterways* Federal Highway Administration, Washington, DC pp 1–111
5.32 RILEM, CEB, IABSE, IASS 1982 *Proceedings of the Inter-association Symposium on Concrete Structures under Impact and Impulsive Loading* Bundesanstalt für Materialprüfing, West Berlin, 2–4 June 1982
5.33 Helminger E 1978 Der absolute und spezifische Materialaufwand für Brückentragwerke aus Spannbeton im Zuge neuzeitlicher Strassen *Bautechnik* **1**: 5–15; **3**: 83–91; **4**: 133–43
5.34 Liebenberg A C 1966 Arch action in concrete slabs (CSIR Research Report 234) *National Building Research Institute Bulletin* no 40, Pretoria, South Africa, p 35
5.35 Rüsch H, Hergenroder A 1961 *Influence Surfaces for Moments in Skew Slabs* Cement and Concrete Association, London

5.36 Krug S, Stein P 1961 *Influence Surfaces of Orthogonal Anisotropic Plates* Springer Verlag, Berlin 193 pp

5.37 Pucher A 1964 *Influence Surfaces of Elastic Plates* Springer Verlag, Vienna, New York 93 charts

5.38 Molkenthin A 1971 *Influence surfaces of Two Span Continuous Plates with Free Longitudinal Edges* Springer Verlag, Berlin 165 pp

5.39 Homber H, Trenks K 1962 *Drehsteife Kreuzwerke: Formeln und Zahlentafeln zur berechnung von Tragerrosten Orthotropen Platten und Zellwerken Systeme mit veränderlicher Trägerhöhe* Springer Verlag, Berlin 318 pp

5.40 Rüsch H 1967 *Berechnungstafeln für schiefwinklige Fahrbahnplatten von Strassenbrücken* Wilhelm Ernst, Berlin, 89 pp

5.41 Homberg H, Ropers W 1965 *Fahrbahnplatten mit veränderlicher Dicke* Springer Verlag, Berlin 104 pp

5.42 Leonhardt F, Mönnig E 1975 *Vorlesungen über Massivbau* vol 2 *Sonderfälle der Bemessung im Stahlbetonbau* 2nd edn, Springer Verlag, Berlin, New York, pp 91–8

5.43 Park R Jun 1964 Ultimate Strength of rectangular concrete slabs under short-term uniform loading with edges restrained against lateral movement *Proc. Inst. Civil Eng.* **28**: 125–50

5.44 Braestrup M W Jun 1980 Dome effects in R.C. slabs: rigid plastic analysis. *Proc. ASCE, J. Struct. Div.* **106** (ST6): 1237–62

5.45 Seidel O 1973 Die Dywidag-Spannbetonkontaktbauweise *Beton- und Stahlbetonbau* **68**: 257–64

5.46 Maisel B I, Roll F 1974 *Methods of Analysis and Design of Concrete Box Beams with Side Cantilevers* Cement and Concrete Association, London, Technical Report 42.494, 176 pp

5.47 Liebenberg A C, Trümpelmann V, Kratz R D 1978 *The Design and Analysis of the New Highway Bridge over the Gouritz River Gorge in South Africa* (Separate publication of CSSA distributed at 8th Int. Congr. Fédération Internationale de la Précontrainte, London, 1978)

5.48 Taylor F W, Thompson S E, Smulski E 1948 *Concrete: Plain and Reinforced* vols 1 and 2, Wiley, New York, 969 and 688 pp

5.49 Deere D U, Hendron A J 1966 Design of surface and near surface construction in rock. In *Proc. 8th Symp. Rock Mech.* Minneapolis, Sept 1966

5.50 Crouch S L 1976 *Analysis of Stresses and Displacements around Underground Excavations: an Application of the Displacement Discontinuity Method* University of Minnesota Dept Civil and Mineral Engineering

5.51 Poulos H G, Davis E H 1973 *Elastic Solutions for Soil and Rock Mechanics* Wiley, New York

5.52 Candrlic V Jan 1981 Zur weiteren Entwicklung des Freivorbaus von weitgespannten Massivbogenbrücken *Beton- und Stahlbetonbau* **76**: 1–6

5.53 Miyazaki Y 1976 Design and construction of the Hokawazu Bridge. In *Prelim. Rept. 10th Congr. Int. Ass. Bridge Struct. Eng.* Tokyo, Sept 1976 pp 21–6

5.54 Candrlic V 1981 Cantilever construction of large concrete arch bridges in Yugoslavia *Concr. Int.* **3** (8): 43–7

5.55 Liebenberg A C Oct 1967 The longspan across the Van Staden's Gorge *Civ. Eng. Contract. (South Africa)* 18–27, 45

5.56 Wössner K, Gebhardt H, Schakel R, Wörner H Oct 1979 Die Talbrücke Rottweil-Neckarbing im Zuge der A 81 BAB Stuttgart–Westlicher Bodensee *Beton- und Stahlbetonbau* 117–26

5.57 Liebenberg A C, Trümpelmann V, Kratz R D 1982 *The Largest Concrete Arch in Africa* Separate publication by CSSA distributed at 9th Int. Congr. Fédération Internationale de la Précontrainte, Stockholm, 1982).

5.58 De Ing G H (Presenter) 1979 Il Ponte di Brotonne Sulla Senna tia Rouen e le Havre *L'Industria Italiana del Cemento*: **9** 499–518

5.59 Podolny W Jr and Scalzi J B 1976 *Construction and Design of Cable-stayed Bridges* Wiley, New York, 506 pp

5.60 Leonhardt F, Zellner W, Svensson H 1980 Die Spannbeton-Schrägkabelbrücke über den Columbia zwischen Pasco und Kennewick im Staat Washington, USA *Beton- und Stahlbetonbau* **2**: 29; **3**: 64

5.61 Leonhardt F 1978 The Columbia River Bridge at Pasco–Kennewick, Washington, USA. In *Proc. 8th Int. Congr. Fédération Internationale de la Précontrainte*, London, 1978, part 2 pp 143–53

5.62 Mathivat J The Brontonne Bridge. In *Proc. 8th Int. Congr. Fédération Internationale de la Précontrainte*, London, 1978, part 2, pp 164–72

5.63 Walter R, Luigi R 1981 Cable-stayed bridges. In *Proc. Symp. Modern Trends in Bridge Design and Construction* Concrete Society of Southern Africa, Johannesburg, July 1981

5.64 British Standards Institution *BS 5400: Parts 1–10: 1978–1984* Steel, concrete and composite bridges

5.65 Department of the Environment 1971 *Provisional Rules for the Use of Rubber Bearings in Highway Bridges* HMSO, London, pp 3–18

5.66 International Union of Railways 1969 *Code for the Use of Rubber Bearings for Rail Bridges* 3rd edn, Paris, UIC Code 772 R

5.67 Lee D J 1971 *The Theory and Practice of Expansion Joints for Bridges* Cement and Concrete Association, London, 65 pp

5.68 Leonhardt F, Mönnig E 1975 *Vorlesungen über Massivbau* vol 2 *Sonderfälle der Bemessung im Stahlbetonbau* 2nd edn, Springer Verlag, Berlin, New York, pp 91–8

5.69 MacDonald M D 1974 *Waterproofing Concrete Bridge Decks: Materials and Methods* Transport and Road Research Laboratory, Crowthorne, Berks, UK, TRRL Report 6.36, 14 pp

5.70 Schlaich J, Schäfer K 1984 Towards a consistent design of reinforced concrete structures. In *Final Report 12th Congress, International Association for Bridge and Structural Engineering* Vancouver, BC, Sept 1984, pp 887–94

5.71 Barrie D S, Mulch G L Sept 1977 The professional CM team discovers value engineering *J. Construct. Div. Am. Soc. Civ. Eng.* **103** (CO3) 423–35

5.72 Miles L D 1972 *Techniques of Value Analysis and Engineering* 2nd edn, McGraw-Hill, New York, 320 pp

5.73 Institution of Civil Engineers 1979 *Conditions of Contract and Forms of Tender, Agreement and Bond for Use in Connection with Works of Civil Engineering Construction* 5th edn, London, 51 pp
5.74 Fédération Internationale des Ingénieurs-Conseils 1987 *Conditions of Contract (International) for Works of Civil Engineering Construction with Forms of Tender and Agreement* 4th edn Lausanne FIDIC 1987 (Reprint 1987 — 2 parts)
5.75 Reynolds C J 1980 *Measurement of Civil Engineering Work* Granada, London, 132 pp
5.76 Juran J M, Gryna F M, Bingham R S (eds) 3rd edn. 1979 *Quality Control Handbook* (3rd edn), McGraw-Hill, New York, pp 2–11, 2–23
5.77 Duncan Wallace I N 1969 *Building and Civil Engineering Standard Forms: A Commentary on the Four Principal RIBA Forms of Contract, the FASS Form of Subcontract, the RIBA Fixed Fee Contract and the ICE Contract* Sweet & Maxwell, London, 473 pp
5.78 Duncan Wallace I N 1974 *The International Civil Engineering Contract: A Commentary on the FIDIC International Standard Form of Civil Engineering and Building Contract* Sweet & Maxwell, London, 197 pp
5.79 Buckland P G *et al* Traffic loading of longspan bridges (TRR 655) Bridge Engineering 2 1978 pp 146–54

6 The Field Inspection and Maintenance of Concrete Bridges

6.1 General

Concrete, if correctly made for any particular circumstance, is essentially a durable material. The durability is determined largely by the quality of the concrete and the environmental exposure. The quality can within limits be made to match the degree of exposure by proper design, selection of materials and admixtures and by good workmanship. Nevertheless all concretes are subject to chemical and physical changes, so that inspections of works at regular intervals are essential as part of maintenance programmes.

In the past the approach of designers has tended to be based on the assumption that concrete bridges, in contrast with steel bridges, had almost limitless durability and consequently required little if any maintenance. For similar reasons some responsible authorities failed to institute adequate inspection programmes and maintenance procedures in spite of the importance of bridges.

More recent experiences have made it clear that, unless very great care is taken in the detailed design and construction of concrete bridges, such notions are misguided. Although soundly constructed reinforced concrete members can last without major maintenance for very long periods approaching the expected (design) life, usually of the order of 100 years, various factors (enumerated below) can cause serious deterioration within the first decade if not attended to in good time.

There is now a more general awareness and understanding of the problems, largely as a result of several very useful seminars or symposia on various aspects of the problem.[6.1–6.3] It is very important that designers have a sound understanding of all potential causes of deterioration and give adequate consideration thereto in the conceptual and detailed design stages (see Chapter 2, Section 2.6), as avoidance is usually better than the cure.

A similar understanding is necessary in carrying out field inspection and maintenance programmes in order to be able to identify and correctly assess any defects or signs of deterioration and apply the most suitable corrective or maintenance procedures. It is important that such programmes be under good management, preferably by a senior engineer, with well-trained personnel consisting of inspectors (engineers and technicans) and administrative officials as a subdivision under the senior bridge design engineer. This does imply considerable expenditure but, bearing in mind the strategic and economic importance of bridges and the costly implications of failures due to neglect, such programmes are undoubtedly cost-effective if procedures are optimized (see Chapter 3, Section 3.2.1).

The first essential of a maintenance programme is a regular inspection routine, preferably prescribed in a bridge inspector's training manual[6.4] to provide guidelines for the training of bridge inspectors as well as in specific handbooks and checklists for reference and use in the field. The procedures may however require adaptation, depending on varying localities and bridge types as well as problems of accessibility. It is consequently essential that such programmes be under the direct control of well-qualified and knowledgeable structural engineers. During the process of carrying out field inspections, all necessary safety precautions should be taken.

Current bridge inventory information and full records of inspections, preferably on standard forms, should be kept. A properly conceived bridge-inspection reporting system is essential; it should preferably incorporate a computerized database. Where defects are observed, full reports should be prepared on the basis of detailed notes and sketches made in the field, and photographs should be taken. These reports, together with all available historical information, should serve as the basis for further studies, computations, statistical evaluations and diagnostic investigations (see Section 6.5). Where any doubts exist, it would be essential for such routine inspections to be followed up by qualified engineers to establish or confirm the true nature of the problem. It may furthermore be advisable to consult the engineer responsible for the original design. A repair procedure, if necessary, can then be prepared.

6.2 The Nature and Causes of the Various Forms of Deterioration and Failure of Concrete Bridges

6.2.1 Primary Causes and Agencies

The primary causes, contributory factors and agencies causing deterioration or failure can be conveniently listed in the following categories:

A. *The influence of the environment*. The forces, actions and effects due to the environment as described in Chapter 2, Sections 2.4–2.7, as well as the corrosion, chemical attack, weathering and other forms of pollution and environmental attack in marine and industrial areas on the materials of construction as listed below.

B. *The inherent properties of the materials of construction*. The capacity to resist the various forms of external environmental actions or attacks and of internal disruption as described below.

C. *The designer*. Conceptual and analytical errors in the design; faulty detailing of the components of the bridge; inadequate contract documentation and/or supervision of the works (see Chapter 5, Section 5.10). Further comments are made below.

D. *The constructor and/or construction supervisor*. Defects in the materials and methods of construction.

E. *The client (owner)*. Over-stringent economical restraints imposed on the designer and/or time constraints on the contractor and lack of adequately managed maintenance programmes.

F. *The authorities prescribing design and construction standards*. Actions and effects not properly understood due to the state-of-the-art or not adequately covered by codes of practice, resulting in faulty design.

G. *Road users and the general public*. Gross overloading of heavy vehicles causing structural damage and fatigue; vehicles travelling at excessive speeds and damaging expansion joints; collision damage to piers and/or balustrades; damage by vagrants.

Historically, causes within all seven categories have separately but often in combination contributed to serious deterioration or damage, and ongoing studies are necessary for a more complete understanding of all the interactions that are involved. Many of the major problems that the engineering profession has faced in the past were caused by one or more of categories A, B, E and F compounded by one or more of the agencies in categories C, D or G. Category F problems relating to design and construction standards are less common today because of recent advances in design philosophy (see Chapter 3) which have been incorporated in most codes of practice. However, in the past there have been numerous examples, including catastrophic collapses, resulting from shortcomings in the state-of-the-art (see Chapter 1, Section 1.2 and Chapter 3, Section 3.2).

In spite of regulations and controls, overloading of vehicles regularly occurs and can contribute to serious overstressing, especially in bridges which do not comply with modern standards. This, if repetitive, can manifest itself in the form of cracks due to fatigue effects which may be exacerbated by defective detailing of steel reinforcement, such as excessive bar sizes or spacing or insufficient anchorage or binding, resulting in excessive bond and/or tensile stresses in the concrete. Differential deformation due to foundation settlements not allowed for in the design is also a fairly common cause of overstressing and cracking of abutments and piers. Although there are many potentially serious defects that fall under category C, such as faulty design based on incorrect assumptions or analyses leading to overstressing of concrete or reinforcing steel, most cases of gradual deterioration seem to relate to category D because of lack of sufficient density of concrete and/or concrete cover resulting in corrosion of steel reinforcement in aggressive environments, especially under conditions of freezing where de-icing salts are usually applied to deck surfaces (see Chapter 5, Section 5.8.5). The use of calcium chloride admixture during construction could also be a contributory factor. Corrosion results in the cracking and/or spalling of concrete due to the bursting effects of the volume increase caused by rust (ferric oxide) formation in the surface of the steel reinforcement. The mechanical action of freezing and thawing cycles can also contribute to deterioration, especially if sodium chloride is present. The rate of corrosion increases due to carbonation (see Section 6.2.2). Leaching of the resultant oxides through cracks or porous concrete to the surface causes unsightly staining. Where structural components are restrained against temperature and shrinkage movements, damage in the form of cracks or spalling may occur. Excessive creep of concrete may lead to unsightly deformations and may in extreme cases lead to the overloading and damage of interconnected components of greatly differing stiffnesses or with different construction histories.

An effect which has only manifested itself in the last three decades is that of alkali–aggregate reaction due to a highly alkaline cement composition reacting with various types of aggregates. The resultant expansion effect has resulted in very serious cracking of concrete, requiring very costly repairs and even partial or total demolition.

6.2.2 Corrosion of Steel Reinforcement

It is important to have a correct understanding of the electrochemical process of corrosion of steel reinforcement embedded in concrete,[6.5] as it is probably the singular cause of most damage to bridge structures. If proper precautions are taken, corrosion effects can be prevented or substantially reduced. Where corrosion does occur, timely repairs are called for. Various methods of counteracting corrosive action can be applied effectively in practice.

Steel reinforcement in concrete is protected from corrosion by a combination of the chemical reactions on the steel surface (passivation) and the environmental barrier provided by the concrete cover. Provided, therefore, that the structure has been properly designed and constructed with adequate concrete cover to the reinforcement to match the severity of the environment, and that the concrete is of the required grade and has adequate density, concrete bridges should not suffer from the detrimental effects of the corrosion of steel reinforcement. The density requirement of the concrete is most important as additional cover by a lower grade or badly compacted concrete will not necessarily remedy such deficiency. The above requirements are, however, in many cases not fully complied with so that a major portion of the damages suffered by concrete bridges can be attributed to corrosion. This applies especially in colder climates where de-icing salts are used on bridge decks, to structures close to or spanning the sea where there tends to be a build-up of salts in a zone above high-tide level, in evaporating/drying cycles where an abundance of oxygen is available, and to industrial areas producing toxic waste. The cost of repairing the damages due to these effects can be enormous as is presently being experienced in several northern countries. If the deterioration is not attended to in good time, the only solution may be partial demolition.

It is consequently essential that the designer should clearly understand the nature, causes and consequences of steel corrosion. The major causes have been mentioned above but there are further contributory causes such as very closely spaced reinforcing and inadequate compaction of concrete resulting in honeycombing. Hair cracks due to permissible tensile stresses on any section do not normally lead to corrosive action as the cracks tend to seal by autogenous healing in humid conditions. Major cracking in excess of that permitted by codes of practice could lead to penetration of moisture and chlorides in solution, with resultant corrosion.

6.2.2.1 The Process of Corrosion

For the corrosion of any metal to occur the following conditions must apply:[6.6]

(a) an electron sink area (anode) must be present at which the electronation reaction (i.e. metal dissolution, oxidation) occurs;
(b) an electronic conductor (in most cases the metal itself) is required to carry the electrons to the electron source area (cathode) where an electronation (reduction) reaction occurs;
(c) an ionic conductor must be present to keep the ion current flowing, and to function as a medium for the electronic reaction.

In the case of reinforcement corrosion in concrete, anodes and cathodes develop either as small, closely spaced areas on the bar itself, or as areas of reinforcement, often up to 3 m apart. The electronic conductor is the reinforcement itself and the ionic conductor or electrolyte is the liquid held within the concrete pores. At the anode, the iron is oxidized to ferrous ions. Where the iron subsequently reacts to form ferric oxide, the change in volume is four-fold. Concrete is highly basic and usually has an adequate supply of oxygen available for the cathode reaction to take place. In concrete, the stoppage of an electric current is brought about most commonly by polarization at the cathode, slowing down the cathodic reaction. Thus, when corrosion of reinforcement can occur because of some interference with the passive film, the condition leading to its inhibition is primarily a very low degree of permeability of the concrete.[6.6]

The process of corrosion of steel is dependent on the relationship of E_H (the standard hydrogen potential) and the pH (hydrogen-ion concentration) and is in turn greatly increased by the presence of chloride ions which may be present in the sand or may penetrate through cracks or low-density concrete. For a pH of about 11, which would be the condition of steel in contact with uncarbonated concrete, and an E_H greater than about -0.5 V, a passive condition would normally prevail. As the pH falls, the tendency for general corrosion to occur increases. In the presence of chloride ions the corrosion may occur despite the high pH of the concrete pore solution. There is then also the likelihood of 'pitting' of the steel due to the development of localized cells. The development of 'pits' in tendons may be due to unfavourable storage conditions prior to installation or to corrosive conditions active in service. At the pit, the site of corrosion, the area is depleted of oxygen and the process is autocatalytic in nature. The pH at the tip of the pit may be about 4, compared with 11 or higher elsewhere on the corroding surface.[6.6]

Preventing access of oxygen (O_2) to parts of a steel bar does not prevent corrosion of these parts, as corrosion may actually be enhanced by the development of differential cells between the oxygen-rich and oxygen-poor sections.

Other factors that may cause differential cells are differences in alkalinity from one area to another and differences in the concentration of chloride Cl-ions. Bleeding, segregation and poor consolidation of concrete may cause differences in environment between the upper and lower surfaces of individual bars. Temperature differences within the concrete may create differences in the electrochemical potential. Repaired areas where, for example, one section of steel is embedded in epoxy and the other in portland cement, may also produce a differential cell. Differences in the steel are caused, for instance, by welds, and ordinary bars connected to zinc-dipped bars may also lead to differential currents.

6.2.2.2 *Carbonation*

The rate of corrosion is influenced by carbonation, which increases with the concentration of carbon dioxide (CO_2) in solution resulting in a los of alkalinity so that the pH falls below the level of 11 required fc passivity. $Ca(OH)_2$ is carbonated to $CaCO_3$, but other cement compounds are also decomposed, hydrated silica, alumina and ferric oxide being produced. Carbonation occurs most rapidly at humidities in the range of 50–75 per cent and is accompanied by an increase in the weight of the concrete and by shrinkage. The rate of penetration into dense concrete is roughly proportional to the square root of time. Corrosion is thought to start in the presence of moisture at a 'critical' 70 per cent relative humidity. The effects of carbonation are increased in combination with SO_2, resulting in sulphation.[6.7,6.8]

6.2.2.3 Permeability and Cracking

The degree of permeability and extent of cracking of the concrete cover to steel reinforcement is critically important in determining the rate of corrosion due to the penetration of moisture, and calcium chloride in solution, which increase the rate of corrosion. This also applies to the penetration of CO_2, as the corrosion of the reinforcement also depends on the carbonation of the concrete and influenced by crack width and concrete cover. However, transverse cracks do not play a major role in increasing corrosion, provided the crack sizes do not exceed specific limits (usually specified by authorities for various degrees of environmental exposure). Longitudinal cracks which are usually associated with corrosion of steel bars may be the result of expansion of the steel as it oxidizes. The permeability of concrete does not depend only on the density of the concrete and that of the cement paste but can be influenced by porous aggregate. Honeycombing and blowholes will increase the rate of penetration.

6.2.3 Alkali–Aggregate Reaction (AAR) in Concrete

Some three decades ago it was observed in several countries that certain concrete structures exposed to moisture penetration developed crack patterns with discoloration of the concrete adjacent to the cracks (Fig. 6.1). Lowly stressed concrete members tended to develop a block cracking pattern, whereas cracks in structural members under unidirectional compression were aligned parallel to the direction of the major principal compressive stress. Subsequent investigations confirmed that this could be attributed mainly to the long-term (usually several years) expansive effects of alkali–aggregate reaction. Actually alkali–aggregate reaction was first recognised by Stanton[6.9] when he investigated the cracking of a concrete pavement in California in 1938. In 1944 Hansen[6.10] realized the possible significance of osmosis in the alkali–aggregate reaction; this finally led to a tie between the chemical processes

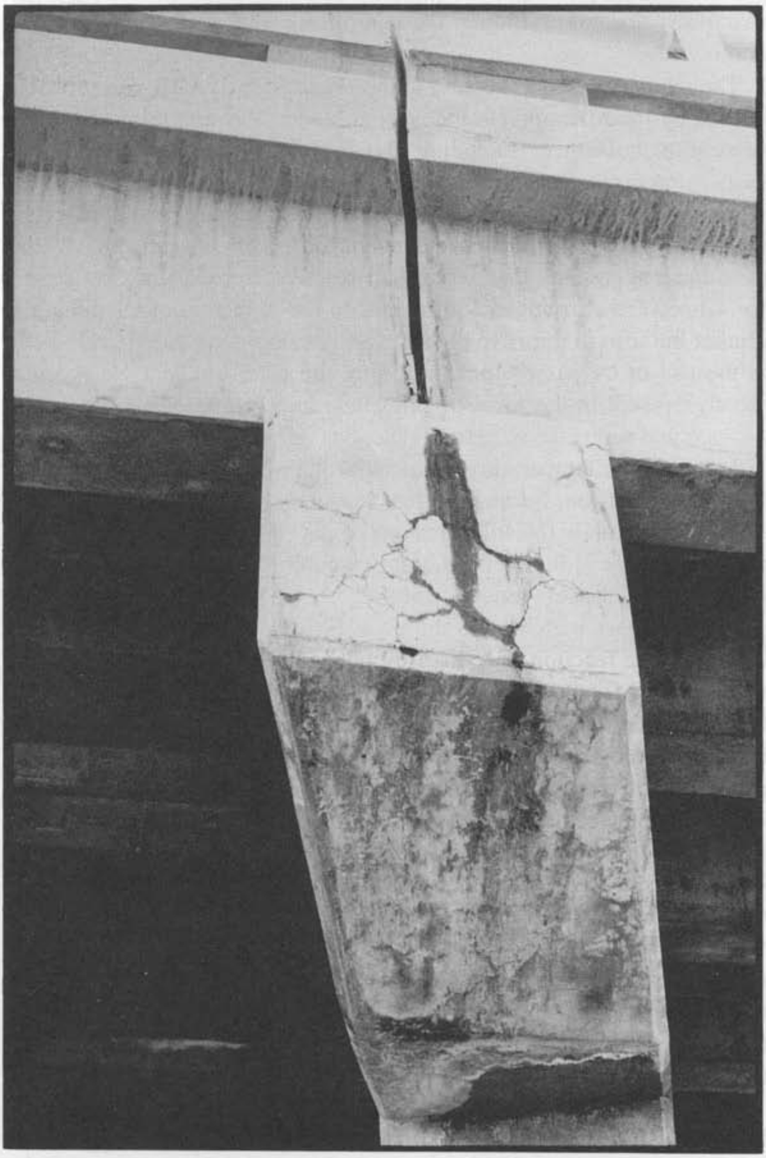


Figure 6.1 Cracks due to AAR in a capping beam under an expansion joint between deck beams (courtesy of Dr R E Oberholster)

and the development of the physical pressure which was required to explain the expansion and disruption which had been observed. It has since been identified and investigated in Europe and various other countries such as Australia, Canada and South Africa. Since 1974 several international conferences have been held on the results of research into the problems of understanding the actual mechanism of the reaction, of establishing rapid and practical methods of testing aggregates for reactivity and of avoiding or controlling the reactions by using admixtures.[6.11]

The relatively recent increase in the occurrence of AAR was probably caused by modifications to the cement clinker burning process used in cement manufacture, including the introduction of energy-efficient suspension pre-heater kilns as well as the use of sulphur-rich fuel. These resulted in an increase in the alkalinity of the finished cement prduct,[6.12] which had the above-mentioned damaging effects in the case of concretes containing aggregates that were alkali-reactive. Subsequently re-designs for kilns were introduced with regard to the reduction of alkalis in the clinker but also to improve the strength development. A partial remedy is the use of by-passes for controlling the alkali content.[6.13] Alkalis, mostly present in the form of sulphates, have a pronounced effect on the rate and sequence of certain reactions during clinker production by influencing the temperature of melt formation as well as its viscosity and surface tension. Because of their relatively low melting point, alkalis remain fluid below the solidification points of other clinker phases and then crystallize in the form of highly soluble sulphates on the surfaces of the major clinker minerals formed.[6.13]

Upon contact with water, the alkali sulphates rapidly dissolve and influence the reaction processes by changing the solution pH and the solubilities of other clinker minerals. Solid products of hydration incorporate alkalis in their structures. This may result in compositional and morphological changes which, in turn, influence the physical properties of fresh and hardened concrete. In particular, the presence of alkalis in the cement paste pore solution may lead to abnormal setting phenomena, decreased strength and reactions with siliceous aggregates. Polished or broken surfaces of affected concrete usually show some of the following features of deterioration (Fig. 6.2):

(a) dark reaction rims around aggregate particles;
(b) white porcellaneous acid-insoluble reaction products;
(c) cracks in mortar; and
(d) loss of bond between mortar and coarse aggregate.

Blight *et al.*[6.14] have shown that alkali–aggregate reaction is aggravated by exposure to alternating cycles of wetting and drying but it may be partially attributed to the mechanical effect of expansion and contraction in addition to the swelling effect of the chemical reaction. Retaining walls and abutments backed by wet earth and beams at expansion joints subjected to cyclic wetting through joints in bridge decks (Fig. 6.1), as well as all exposed surfaces collecting a flow of water or direct rainfall, are particularly prone to cracking. Although it has been suggested that high alkali values result from concentration by leaching of alkali by

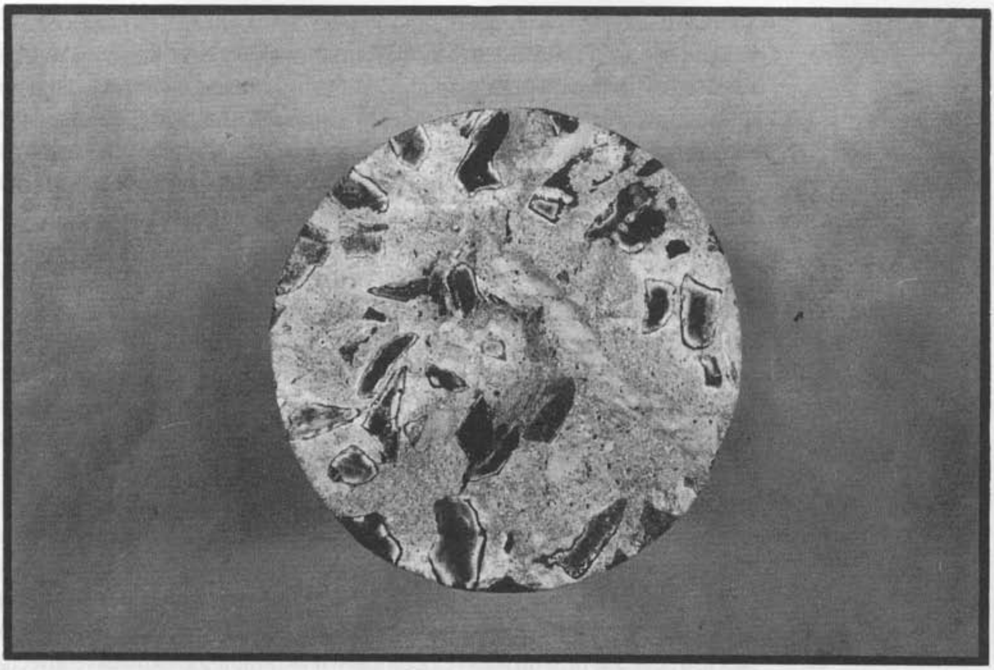


Figure 6.2
Section through concrete core showing the effects of AAR (courtesy of Dr R E Oberholster)

intermittent wetness, which would explain why AAR-deteriorated concrete is often found to have an alkali content that far exceeds that of the original concrete mix, they could find no clear-cut evidence of such effects by leaching in their investigations. Concretes containing highly alkaline cements and reactive aggregates do not appear to deteriorate if permanently dry, and are less susceptible to these constituents if permanently immersed in either fresh or sea water.[6.15] It would seem that the swelling due to alkali–aggregate reaction occurs in moist concrete and that the cracking is due to differential straining caused by localized swelling; this would explain the existence of cracks in surfaces which on average have a lower moisture content than the inner core of the concrete member. It would also explain why totally immersed concrete does not appear to deteriorate to the same extent.[6.16]

There are three types of alkali–aggregate reaction, viz.:

(a) alkali–silicate reaction involving cherts, flints, chalcedony, opal, naturally occurring acid-volcanic glass;
(b) alkali–carbonate rock reaction for which the texture of the aggregate must be such that larger crystals of dolomite are surrounded by, and scattered in, a fine-grained matrix of calcite and clay; and
(c) alkali–silicate reaction ascribed to some phyllosilicates which react to form swelling gel.

Severe reactions have been reported in South Africa where certain metamorphosed slates, e.g. Malmesbury hornfels, have suffered severe swelling if used with highly alkaline cements, causing extensive damage to structures exposed to moisture.[6.6] Although the critical amount of alkali content varies depending on the particular aggregate, the reactive effect normally does not occur for cements with an alkali-content of less than 0.6 per cent Na_2O equivalent, calculated as

$$\% \ Na_2O + (0.66 \times \% \ K_2O)$$

The calculated alkaline content must include any additional content introduced in the form of alkali salts, fly ashes or other pozzolanic materials. Various authorities use different upper limits, depending on the local factors. Most allow up to 0.6 per cent, although expansions have been observed with an Na_2O equivalent as low as 0.4 per cent. Standard testing procedures have been developed by various authorities in an attempt to establish a basis for evaluating the potential reactivity of aggregate or of cement–aggregate combinations. The interpretation of these results usually requires experience and careful judgement.

The alkali content of cements can be effectively reduced by the use of admixtures such as blast-furnace slag, calcined shale or pulverized fuel ash, certain of which have been shown to inhibit the expansive effects by pozzolanic or other reactions with the hydroxide ions.

On the other hand, there are many known cases where the aggregates used and the high alkaline content of the cement seem to indicate the likelihood of severe reaction but no adverse effects have been observed over long periods. The reactive process is highly complex and as yet not perfectly understood. There appears to be an elusive factor involved in the reaction mechanism that is still unkown.

6.2.4 Chemical Attack

Concrete bridges near industrial areas may be subjected to groundwater contaminated by factory effluent or landfill dumps containing chemicals such as sulphates and chlorides in concentrations that may attack the cement mortar or steel reinforcement in foundation concrete. These chemicals may cause deterioration of the surface zones of the concrete or penetrate through cracks or by permeation and cause more deep-seated damage to the concrete and/or the steel reinforcement. The tolerable level of sulphate concentrations in relatively dry, well-drained soils is about four times the acceptable limit when the sulphates are in the form of a solution in groundwater.

6.2.5 Shrinking Aggregates

Certain aggregates which otherwise would be suitable for concrete have been found to undergo excessive shrinkage, causing shrinkage movements in concrete up to seven times greater than those which occur in similar concretes containing normal aggregates. The rock types involved are shale or sandstone-like materials and frequently appear to be hard and sound to the degree that no engineer would condemn their use on appearance alone. Normal laboratory tests such as those prescribed in most specifications do not reveal the unsuitability of many shrinking aggregates and special tests directed particularly at the evaluation of shrinkage characteristics are necessary to detect them.[6.17]

Petrographic and petrological studies using X-ray methods on aggregates from the Karoo system in South Africa[6.18] have indicated that illite, chlorite and montmorillonite are present. From various aggregates tested, the conclusion was reached that although montmorillonite can indeed cause dimensional changes of this type it is not essential for clay in the aggregate to be of the unstable lattice type in order to produce the effects observed.

Concretes made of shrinking aggregates deteriorate and undergo excessive deformations due to internal cracking around the aggregate particles. As the cause is deep-seated there is no ready method of repair, apart from using additional stiffening members. The use of aggregate or sands that undergo excessive shrinkage should therefore be avoided.

A valuable indication of whether a particular aggregate is satisfactory may be provided by examination of structures which are known to have been built using the aggregate. Where there is a record of successful use over a period of at least five years, there is no valid reason for prohibiting its use in similar structures under similar conditions of exposure. Field and laboratory tests to identify shrinking aggregates have been developed.[6.19]

6.3 Field Inspection

6.3.1 General

The maintenance of concrete bridges can be best assured by a well-organized and managed inspection programme which is systematically carried out at regular intervals by suitably qualified inspection personnel.[6.8,6.20] The nature and frequency of the inspection procedures will depend on the importance, type and age of the bridge or bridge component, the materials of construction, the environmental conditions and the traffic characteristics. The intervals between inspection should not exceed two years but bridges in a poor state of maintenance or with potential or known deficiencies, e.g. bridges posted for a weight limit, are to be inspected more frequently.

The problems encountered in this work are numerous, variable and often complex. Sound judgement and/or specialized knowledge and skills are very often necessary for the proper evaluation of a problem. A bridge inspector should therefore never hesitate to consult a second or preferably an expert opinion on a problem with which he is not familiar.

Inspections should preferably coincide with the periods of the year which offer the most favourable conditions for thorough work. Substructures of bridges over rivers should preferably be inspected at times of low water and structures requiring high climbing during those seasons when high winds or extreme temperatures are not prevalent. Joints or bearings that are affected by thermal movements and the effect of de-icing salts (for example) should be inspected at the times most suited to observe the effects. It is also important that probable defects be anticipated so that preventive maintenance may be carried out in good time.

6.3.2 Inspection and Appraisal Procedures and Strategies

Field inspections can be categorized under routine inspections forming part of a standard maintenance programme, or special inspections required due to an immediate need resulting from accidental or suddenly perceived serious damages, or as part of an investigation into assessing the capacity of the bridge to carry heavy or abnormal loads. The inspection procedures can furthermore be subdivided into (a) standard procedures which would apply to almost all bridges and components, (b) specific procedures that apply only in special cases and (c) those procedures that cannot be clearly prescribed and consequently require a clear understanding by the inspectors of the function and properties of bridge components in order to be able to recognize a sign of deterioration or malfunction and to improvise procedures. These procedures would apply to unusual phenomena that have not been experienced previously.

The above inspection procedures are usually carried out in stages, namely:

(i) reconnaissance;
(ii) preliminary or exploratory inspections to determine the general condition of the bridge, to identify components that require more thorough and detailed investigations and to establish what equipment may be required for the main or principal inspection; and
(iii) the main inspection involving systematic procedures of investigation, including where necessary *in situ* testing and sampling of materials followed by laboratory testing and analysis.

The preliminary investigations are very important and must be done by knowledgeable and experienced inspectors who should not hesitate to consult senior engineers when they are in doubt about the significance of some abnormality. The results of the main inspection and tests and of any subsequent laboratory tests, together with any supplementary analytical work, will form the basis of the appraisal and the report with recommendations for actions.

Authorities (some subject to explicit governmental legislation) usually prepare detailed inspection procedures applicable to bridges under their jurisdiction[6.8,6.20] in which systematic sequences of investigations are spelled out for various types of bridges and the components. These investigations are based on the search for symptoms of malfunction and damage, e.g. cracks and spalling of concrete, leaking and leaching and discoloration of concrete, corrosion of steel and rust staining, chemical attack, excessive deformation or vibration under traffic, and differential movement or settlement of piers and abutments. These investigations should also include the verification of the soundness and proper functioning of moving or articulating parts such as expansion joints, bearings and hinges which may deteriorate or become obstructed by corrosion or extraneous material. Any loose anchor bolts should be tightened. Some of the characteristics of elastomeric bearings require particular attention. They should be examined for splitting, tearing or cracking of the outer casting and for bulging and distortion caused by excessive compressive forces and/or shear movements or excessive rotation. In the case of low-friction sliding material such as Teflon (PTFE) on stainless steel, tests should be done to establish that movement is taking place at the sliding surface. Furthermore, the surface levels and smoothness of decks and approach road surfaces should be verified to avoid excessive impact effects. The condition of the asphaltic or other types of road wearing surfaces, sidewalks and kerbs should be investigated for signs of deterioration, unevenness or settlement. Components such as bridge and barrier railings and signposts or columns that may be subjected to impact of vehicles should be carefully scrutinized. Outer beams of underpass bridges should be inspected for impact damage from high vehicles. Interior surfaces of box-girders should be checked for defects. The effective functioning of all drainage systems including weep holes should be verified and, in the case of bridges over rivers, the adequacy of the waterway should be observed during flood periods when ice or debris may present a serious threat to the structure. Road embankments and river-bank protection devices should be checked for soundness, especially where changes in the channel flow have occurred.

Signs of scouring or undercutting action in the vicinity of foundations should be noted for further investigations where considered necessary (see Chapter 2, Section 2.4.5). Observations of the effect of stream flow and wave action on the bridge and its approaches should be made for comparison with design assumptions in order to anticipate possible flooding problems. Erosion of banks or levées should be investigated. The above items are not intended to be an exhaustive list but only typical of the necessary investigations.

The use of check lists of inspection items with coding and field books or standard forms for making notes and sketches supplemented by photographs is advisable. Where dimensions and levels are required, suitable surveying equipment is essential. Where observations of static or dynamic deflections or strains due to traffic or other actions are required, the sophistication of the equipment will depend on the nature of the problem. Standard sampling procedures of materials should be applied where possible.

Safety precautions for inspectors and workmen are of the utmost importance: long-span and high bridges, where feasible, should have built-in provision to facilitate inspection and maintenance work, e.g. box-section arches and girder bridges should have access to all interior compartments, with adequate means of climbing where steep inclines or great heights are involved. Permanent ladders and walkways are desirable, but ropes or chains attached to hooks may in some cases be adequate. In the case of confined spaces, pre-entry air tests for oxygen content or the presence of other toxic fumes or gases with approved devices[6.4] may be necessary and mechanical ventilation may need to be applied continuously during occupancy. Inspection of high parts of bridges may be hazardous and adequate scaffolding, ladders, platforms, or a truck-mounted bucket on a hydraulically operated boom should be used. Lifebelts should be worn when working at heights above 6 m (20 ft) or above traffic.

6.3.3 Inspection Reports

Inspection reports are important documents which may serve as the basis of costly repair procedures and may become a legal record to be used in future litigation. The language used should be clear and concise and ambiguous statements should be avoided. However, where some conclusions are uncertain, this should be clearly stated. The information contained in reports is obtained from field inspections supplemented by 'as built' or 'field checked' plans, photographs, measurements and tests carried out during inspection or subsequent tests done in laboratories on samples. It may be necessary to do supplementary calculations in order to assess the reliability of the bridge in its present state.

All signs of distress, failure or defects worthy of mention, as well as the general condition, should be described with sufficient detail and accuracy so that another inspector at a future date can easily make a comparison of condition or rate of deterioration. Photographs and sketches should be used freely as needed to illustrate and clarify conditions of structural elements. Good diagrams are very helpful for future investigations in determining progression of defects and to help determine changes in magnitude.

All recommendations and directions for corresponding repair and maintenance should be included. It is furthermore essential that such reports should be fed back to senior design engineers for review and for further action, especially in the case of category C problems referred to in Section 6.3.2 above. Inspection by inspectors should be supplemented by occasional visits to bridge sites by senior design engineers. This feedback process and subsequent site visits have great heuristic value for design engineers.

Where a group of bridges is inspected, a combined report grouping similar defects together is useful for deploying maintenance resources more efficiently and reducing costs.

6.4 The Maintenance of Concrete Bridges

6.4.1 General

The importance of well-managed maintenance of concrete bridges is self-evident in terms of the strategic and economic importance of bridges. It has a significant effect on the evaluation of the structural utility function (see Chapter 2, Section 3.2.1) in that neglect thereof may greatly increase the risk factors of the function, and vice versa. Apart from matters relating to the statistically expected behaviour, timely maintenance may prevent deterioration by natural processes or even collapse of partially damaged components due to unexpected (low-probability) or accidental causes. The reliability of a bridge is therefore greatly increased by consistent good maintenance.

6.4.2 Organization

Countries with developed road systems ideally should have legislation covering aspects of bridge inspection such as frequency of inspections and qualifications of inspectors. Experiences in many countries, however, have demonstrated that this is not essential for effective bridge inspection. Preference is given to the use of codes of practice and departmental procedures to suit local circumstances, yet it is essential that a definite organizational structure be established with clear definition of objectives, duties and responsibilities. This is especially the case in a type of operation that is in a way passive and therefore requires a high level of commitment. The various functions can be classified under the following headings:

1. Management and general administration.
2. Technical documentation and training of personnel.
3. Bridge inventory data control and reporting.
4. Field inspection and appraisal reporting.
5. Field and laboratory testing.
6. Recommendations in the form of various reports on further testing procedures, repairs and rehabilitation, or replacement and reconstruction.
7. Maintenance instructions.
8. Follow-up maintenance reports and recommendations.

Technical documentation covers inspection training manuals and field inspection and appraisal forms to be used in the field to ensure that inspections are carried out and reported in a systematic manner. Bridge inventories or registers are established by most authorities but are not always used effectively in the management of maintenance. It is important that the documentation should be designed to provide effective interaction between the inspection reports and the permanent bridge inventory. The most satisfactory way, especially for a computer-based filing system, may be to transfer summarized information from the inspection reports to the bridge files, the original reports being kept only for exceptional queries.[6.20] Authorities differ in practice on which items of information should be stored in an inventory. Some use coding systems. The following are items that generally seem to be considered as useful, but some authorities use abbreviated lists:[6.8,6.20]

> Bridge number; bridge name; file number; bridge rating (design codes, traffic loading, width and vertical clearances, and also for bridges over rivers: design return period, design discharge, design freeboard, highest known flood level, design flood level, depth of flow); district or region in which the bridge is located; road or rail class; features intersected (e.g. road over rail, road over road, road over river, etc.); location (route and chainage); trigonometric data; skew; carriageway widths; number of traffic lanes; number of footwalks; number of cycle tracks; overall width; overall length; alternative routes and length of deviation and rating thereof; design authority; construction authority; year of construction completion; bridge configuration; summary of span lengths; types of span and construction materials; deck construction, type of balustrade; roadway width and vertical clearances on and under the bridge (if applicable); type of

abutments and heights; types of piers and heights; original levels of suitable datum targets; types of foundation and founding depths; information on sub-soil founding material; founding capacity and settlement characteristics; type of expansion joints; type of bearings, design loading or current rating; photographs.

Some authorities have supplementary inventory data with a great more detail. Data should also be included covering the history of defects and the repairs carred out. Where bridges have been down-graded due to previous damage this should be noted in the inventory. This should also apply where defects are still under observation prior to repair, in which case projected needs should be recorded.

Authorities that are responsible for a large number of bridges usually have a computer system to manage the condition monitoring, management reporting and maintenance of these bridges.

6.5 The Appraisal, Testing, Repairing and Upgrading of Concrete Bridges

6.5.1 Assessment Procedures

It may be necessary to make various assessments of certain properties of a bridge structure during or after construction or when in service. The reasons may be that defects or symptoms of deterioration have been observed during inspections or that damage or partial or total failure has arisen as a result of one or more action effects. Typical examples of damage resulting from action effects are cracking of concrete and fatigue of steel reinforcement due to overloading of heavy vehicles; impact damage due to vehicles in collision with piers or balustrades or overhead beams; undermining of foundations and piles due to scour action of rivers in flood as well as the washing away of embankments; corrosion of steel reinforcement in severe environments due to lack of concrete cover and/or insufficient density of concrete; cracking of concrete due to alkali–aggregate reaction; and cracking of concrete members due to shrinkage or temperature effects.

Gross errors due to the designer or the contractor, or a combination of both, have been the cause of several catastrophic collapses but also of many more lesser failures which nevertheless have caused serious problems and resulted in considerable inconvenience and financial loss.

The objectives of such assessments would be any or all of the following: to establish the nature, causes and extent of the observed defects, deterioration or damages and the implication thereof expressed in terms of probable repair or replacement costs, consequential risks, effect on traffic or river flow and any other relevant matters such as liability of the parties concerned. In any particular case there may be several alternative strategies of correcting or reducing the defects or of accepting the defects even at an increased (but permissible) risk if such action should lead to optimum structural utility under the pertaining circumstances (see Chapter 3, Section 3.2.1). In making assessments it should be borne in mind that certain components of the partial safety and materials factors can be reduced under specific circumstances as follows: where quality control of the mixing, casting, placing and curing of concrete was of exceptionally high standard and samples were tested systematically, the probability of material failure would be reduced below that on which the code calibration was based. Furthermore, where below-strength concrete was identified by cube tests and consequently rejected, it could be argued that the theoretical probability of failure, as expressed by the convolution integral of the probability density function of the load effect and the cumulative distribution function of resistance, was substantially reduced (see Figs 3.1, 3.2 and 3.4). Although the code calibrations are done using the safety index concept, a reduced standard deviation would imply a similar reduction of the probability of failure or an increased capacity to resist load. On the contrary, damage[6.21] suffered by a structure could result in an increased probability of failure (reduced reliability) if the damage is at a critical position in a critical member. In carrying out an assessment of the load capacity of the damaged structure, a normal deterministic analysis taking into account the allowable increased strength of concrete due to ageing could give a result comparable with prescribed standards. A detailed analysis of the damaged member would permit a reduction in partial strength factors applied to the parts examined, if thereby particular uncertainties incorporated in the partial factors have been reduced. Where redundant members partially or totally compensate for the resistance of the damaged member, this could be taken into account. Furthermore, the maximum probable loads may be patently less than or more than design code requirements.

The above considerations are particularly relevant where a decision has to be taken as to whether continued use by traffic can be permitted until repairs have been carried out or whether temporary procedures such as restricted lane usage or propping of the affected portion of the structure are necessary.

Research is currently being done in various countries on methods of rating and evaluating the remaining life of bridges. The statistical information gathered during routine maintenance inspections will play an important role in the updating of such assessments. It is, of course, no precise science, as many aspects cannot be quantified precisely. The greatest benefit that can possibly be derived from such studies is that appropriate measures of rehabilitation can be taken in good time to prolong the useful life of the bridge.

Where the need exists to upgrade an existing bridge, assessments would likewise be required to include the establishment of the relevant implications of various alternative methods of upgrading the bridge in comparison also with partial or total replacement if at all feasible.

The ability to observe visually is probably the most important attribute needed for the assessment of the 'state' of a bridge, and serves as the basis for determining the procedures and techniques of measuring or testing that need to be done to prove or assess the nature, magnitude and implications of defects.

It is further essential that the inspector should be familiar with the design and construction of the bridge, and prior to inspections he should familiarize himself with any details of the construction methods and material that may be available.

6.5.2 Testing and Sampling Techniques

Various testing procedures have been used with varying success and research towards improved methods is an on-going process. Standardized methods[6.54] have been developed but situations may require innovative testing procedures. Frearson has given a very good summary[6.22] of testing and sampling techniques, as follows:

(a) *Test locations*. The decision 'where to test' is never easy: experience and judgement are essential. In many cases, however, various factors, both technical and non-technical, may effectively limit the choice of test locations. Test data are needed to characterize the concrete associated with each different failure condition, and with that from apparently unaffected 'control' areas from each part of the bridge, e.g. piers, abutments, beams, decks, parapets, etc.; also the influence of different environmental conditions, e.g. exposed and sheltered areas and any subject to rundown of water. Access limitations, especially in the exploratory phase, may restrict the availability of locations both for sampling of the concrete itself and for other less destructive tests.

(b) *Samples for test*. Obtaining test samples to characterize the concrete is an essential part of any investigation. Sampling of a structure for subsequent laboratory testing is generally considered to be a destructive technique, although many client authorities do not regard drilling or coring as destructive techniques as the structure is reparable. The samples commonly obtained during bridge investigations include:

  (i) Surface deposits. Salt deposits, efflorescence and exudations can be removed from the structure, and chemically analysed to provide information on the reaction mechanisms that may have occurred within the concrete.

(ii) Spalled concrete. While causing no further distress to the structure, samples of spalled concrete can often be readily removed and used for preliminary checks on chloride content, carbonation or general concrete composition. Care has to be taken when interpreting the results as spalled pieces of concrete may not be fully representative.

(iii) Drilling dust. Concrete dust samples collected incrementally by use of a rotary percussion drill are frequently taken to determine, for example, chloride penetration profiles. Drill holes may also be used as locations for determination of carbonation depths, the edge of the hole being chipped to provide a broken surface for application of phenolphthalein indicator solution.

(iv) Lumps. 'Lump' samples broken off from the structure may be taken for preliminary indicative testing. Sawn 'lump' samples are rarely taken, as coring is generally a more convenient technique.

(v) Cores. Cores provide the most convenient method of bulk sampling hardened concrete. A range of diameters from 150 mm, through the more usual 100 mm size and down to 50 mm and smaller, can be used. Core samples are usually suitable for strength assessment and a wide range of other tests. Not all sizes of core may be suitable, or indeed sufficiently representative for all the tests outlined below. However, the tests commonly used (together with appropriate references to methods) include:

visual and petrographic examination;[6.23,6.24] AAR assessments and expansion tests;[6.25] cement type and content; water/cement ratio and original water content;[6.26] absorption/porosity/permeability;[6.27–6.29] compressive strength and density;[6.23,6.30–6.32] carbonation profiles;[6.33] chloride penetration profiles.[6.34,6.35]

In addition, when appropriate:

air-void content/spacing factor;[6.36] modulus of elasticity (static[6.37] or dynamic);[6.27] drying shrinkage and wetting expansion;[6.27] coefficient of thermal expansion.

Samples of reinforcing steel can be obtained for examination and further testing when appropriate, by deliberately drilling through, rather than avoiding, the reinforcing steel.

With all samples, due consideration has to be given to their representability.

(c) *Non-destructive test methods*.[6.38] There is a considerable range of 'non-destructive' tests, very few of which are, in fact, entirely non-destructive. Although many might regard the drilling of a core as non-destructive, in some circumstances even the indent from a rebound hammer or a grease mark from a UPV contract point could be said to be destructive, if it resulted in an aesthetically unacceptable feature.

The 'non-destructive' tests and techniques most commonly employed on bridge investigations are summarized below. Details of the techniques are not given but sources of reference are indicated, and a comprehensive list of test methods is given in the BS 1881: Part 200 series.[6.39]

(i) Visual condition.[6.40,6.41] Although frequently overlooked, an expert appraisal of visible features is one of the most useful 'non-destructive' techniques. This may vary from a brief visual appraisal to a full inspection.

A record of the condition of all areas of the structure is normally an essential preliminary stage prior to planning the remainder of the investigation.

(ii) Steel location and cover.[6.42] The various electromagnetic 'covermeter' instruments, in addition to measuring cover depth, provide steel location data for structural assessments, for avoidance during coring (unless a steel sample is required), and allow location of contacts with the steel for half-cell potential testing.

(iii) Half-cell potential.[6.43,6.44] Half-cell potential surveys can be carried out rapidly to identify areas at risk from corrosion.

(iv) Resistivity.[6.43,6.45] The resistivity of the concrete can provide an indication of its moisture condition, *inter alia*, and hence the possibility of any corrosion being able to proceed.

According to Millard,[6.46] the measurement of concrete resistivity promises to be a valuable tool when used in conjunction with potential mapping, in the assessment of the presence and severity of reinforcement corrosion problems. Whereas half-cell potential mapping techniques have been used successfully to indicate regions where steel reinforcement is corroding unseen beneath the surface of concrete structures, the mapping of the electrical resistivity of the concrete can be used in conjunction with the potential method to assess the severity of the corrosion problem in addition to the location of corroding regions.

Non-destructive tests less commonly required, but of value in certain circumstances where specific comparative data are required, include:

(v) Rebound hammer.[6.31,6.42] The rebound hammer may be useful as a comparative indicator of strength in concretes of similar age. It can be used, for example, to show differences in quality between the top and base, as cast, of precast units and to identify their 'way up' in the structure, so that appropriate correction factors are used when calculating '*in situ* cube strength'[6.30] from cores. Estimation of strength values may be possible if sufficient calibration is carried out against, for example, core specimens.

(vi) Ultrasonic pulse velocity (UPV).[6.31,6.42] Although often used for relative strength assessments, the technique is more valuable for comparative assessments of quality and detection of undercompacted and honeycombed areas.

(vii) Absorption, initial surface absorption and permeability. The initial surface absorption test (ISAT)[6.27,6.28] can be used to provide an indication of concrete surface quality, although the technique may be less useful with older concrete. Whilst only measuring 'surface' permeability, this can be of value in assessing the critical cover area.

A range of absorption and *in situ* permeability tests have been reviewed by a Concrete Society Working Party.[6.29]

(viii) Resonant frequency/acoustic emission.[6.47] The analysis of vibrations induced in the structure, or of the response to various sound signals, is probably one of the most significant recent test developments. Whilst these methods may not be readily usable at present, they should prove to be valuable when experience in interpretation techniques has been developed.

Sonic (pulse–echo) integrity testing equipment is available for the testing of piles. It is a powerful method of quality control of piles but can also be used on cast *in situ* piles to check the pile length and to detect major defects. The tests are conducted by pressing a transducer on to the top of a pile while striking the pile head with a hand hammer. The field computer registers the impact of the hammer followed by the response of the pile and shows both on the display. The original can be stored by the operator in a memory.

Many other non-destructive methods of test may be appropriate in certain circumstances and include various strength tests such as penetration resistance, pull-out, pull-off and break-off tests,[6.48] and radiography.[6.42]

6.5.3 The Full-Scale Load Testing of Concrete Bridges

Various situations may arise where it may be necessary to carry out the full-scale load testing of bridges:

(a) bridges to be subjected to increased loads in excess of the design loading;
(b) bridges that are suspected of having a reduced load-bearing capacity due to under-reinforcement, damage or deterioration;
(c) bridges that have been repaired or rehabilitated after severe damage or partial failure and where the effectiveness of the repairs is not certain.

Various procedures, alone or in combination, may be used, depending on the circumstances, to simulate equivalent static and dynamic loadings approximating to the load cases required to prove the resistance of particular structural members or combinations thereof:

1. Using arrangements of jam-packed loaded heavy road vehicles or locomotive trains (as the case may be).
2. Using kentledge, deep layers of sand or water tanks.
3. Using heavy vehicles travelling at speed to study the effects of impact forces. An artificial hump on the road surface or a plank may be used to generate the impact force.
4. Using jacking forces (hydraulic rams) applied in a manner to simulate load effects on particular members. Subsoil or rock anchors provide a convenient means of providing the reactive force.

These tests should be carried out in accordance with the provisions prescribed by the relevant Bridge Authority or applicable code. Sophisticated mechanical and electronic equipment is available to record deflections, extensions and strains in both concrete and steel reinforcement. The most recent developments use laser equipment for the very accurate recording of deflections. It is usual to apply the loading in small increments, carefully monitoring deflections and strains in order to observe any deviations from the expected behaviour in good time. The maximum applied loading in the case of non-destructive tests would depend on the purpose of the test but should not be such as to induce permanently open cracks in the concrete in excess of the permissible design crack width (usually between 0.10 and 0.25 mm depending on environmental factors). The loading at which this would occur would depend on the design details but in any case the maximum loading applied should not have a larger effect than that of 85 per cent of the factored design loads. Where hydraulic rams with cable anchors are used, precautions should be taken against possible cable failure and the sudden release of energy. Where kentledge or vehicles are used, precautions should similarly be taken and where necessary a temporary system of shoring provided with a gap of only slightly more than the maximum anticipated deflection so as to prevent total collapse should there be failure during the test. Where the type of possible failure anticipated is not of the sudden 'brittle' type due to shear or crushing or due to buckling, the load–deflection or load–strain diagrams recorded as the test proceeds should give an indication of incipient failure by the rate of change of the inclination of the graph.

6.5.4 The Repair or Rehabilitation and Upgrading of Concrete Bridges

6.5.4.1 General

Damage to concrete bridges may not only be of a material or structural nature but may have adverse effects on the aesthetic appearance. Whereas it should be sought to repair the structure in the most effective manner, care should be taken that the method of repair does not aggravate the situation. Where upgrading is involved this may pose a major problem and great ingenuity may be necessary in some cases to apply strengthening systems that do not deface the overall aesthetic appearance.

Considerable literature on the repair of concrete bridges has been produced in recent times,[6.1,6.2,6.5,6.49,6.50] and only brief descriptions are possible within the scope of this book. Methods of repair and improvement of a few typical cases of deterioration or failure are given below under various categories. The above references should be consulted for further reading. There are, however, a few general principles and rules that may usefully be borne in mind when undertaking the planning of repair procedures:

1. During the repairing of a bridge all due measures of ensuring the stability and safety of the works should be taken and notices, warning lights, barriers and fences provided to give the public due warning and protection. Where necessary the traffic should be diverted to alternative routes.
2. Certain members of the damaged structure may be in a state of overstress, in which case stress relief procedures should be applied with due care.
3. Additional temporary supports should be applied where necessary to avoid any disastrous collapse or local failure. Special precautions should be taken where unbonded prestressing cables are involved.
4. Surfaces which are to be treated in any manner, e.g. grouting,

shotcreting, epoxy resin application or concreting, should be cleaned and prepared according to the specific requirements.

5. All existing reinforcing steel should be cleaned and carefully rebent and correctly positioned where necessary, and additional steel should be accurately placed with adequate concrete cover and correctly lapped and spaced.
6. Where new concrete is added on to the existing concrete, the contact surfaces should be cleaned by means of hand- or (preferably) sand-blasting pneumatic tools, and excessively cracked portions should be removed. The prepared surfaces should be wire-brushed or water-jetted clean of all dust and other dirt. The design of the repair work should, where necessary, make provision for the effects of differential shrinkage and creep between the old and new concrete. Suitable methods of bonding should be applied using keying with or without steel dowels.
7. All repaired elements should, where necessary, be analysed in composite action with the added parts and the interacting or transfer forces carefully analysed to the extent that it is practicable, bearing in mind the relative ages of concrete and the states of stress and strain of the steel and concrete in the various parts.
8. All repair work should be neatly finished, cured as required and protected against damage until hardened.
9. Where temporary supports or adjuncts or jacking forces have been applied, due care should be taken during the removal and stripping stages to ensure the correct transfer of stress to the new parts.

6.5.4.2 The Repair of Damaged Concrete

Under this heading we are referring to honeycombed portions, chipped edges or superficial damages by the impact effect caused by a vehicle collision as well as structural cracks due to load or more serious damage due to overloading, impact or explosions.

Honeycombed or low-density portions of concrete, if not too extensive, can be reinstated effectively by cutting out the low-density portions with pneumatic or hand-cutting tools and replacing with a suitable mix of concrete or mortar filler. Small volumes can be filled with a semi-dry mortar packed in tightly by light ramming with a flat-ended tool. Reputable and highly specialized repair mortars and renderings for repairing cavities or chipped surfaces of concrete are available and should be applied in accordance with the supplier's instructions. Large volumes should be recast in a concrete matching the original concrete and additional reinforcement provided if necessary. The arrangement of shuttering and the casting procedure should be such as to allow all contact surfaces to be well compacted and for all air to escape. This can be achieved in difficult cases by using a submerged pipe (tremie) to ensure that the void fills from the bottom up, or shaping the formwork in such a manner that the opening through which the concrete is placed is higher than the cavity. Excess concrete can subsequently be removed by chipping and the surface ground smooth; alternatively, a gap of a few centimetres can be left between the upper surface of the new concrete and the matching surface of the parent member and subsequently caulking the gap with a semi-dry cement mortar.

Where the concrete has been damaged by impact force or explosion with portions chipped out and otherwise badly cracked, a decision has to be taken as to the extent of the cutting out that is required as against repairing the cracks and retaining portions in the damaged area. Failing tests [see Section 6.5.2(c)], good judgement is essential and the criterion is usually how effectively the cracked or splintered portions can be repaired by grouting or injection of epoxy resin. Crack repair is dealt with below.

Where a substantial part of a structural member has been damaged it may be necessary to reinstate the original stresses and deflections approximately before the repair work is carried out; for example, where a large portion of the compression flange of a prestressed concrete member near midspan is missing as a result of an explosion while prestressing tendons are undamaged, the structure should be relieved of dead-load stresses by suitably arranged jacking or other methods of propping to introduce a reversal of deflections. An assessment of the probable stresses in the remaining members of the structure should be carried out in order to plan the reinstatement procedures. When removing concrete under compression and replacing it with new concrete, the effect of creep and shrinkage must be considered to avoid overstressing the existing concrete. After all broken surfaces have been adequately prepared, loose portions chipped away and formwork and reinforcing provided, the missing portion of the flange may be cast.

After gaining adequate strength, the jacking forces should be gradually reduced and additional loading applied to the deck as required to allow some creep and shrinkage to take place towards equalizing the stress in the new and old portions of the compression flange. Structural cracks may be due to any one or a combination of the following effects:

(a) shrinkage of concrete;
(b) temperature expansion or contraction of concrete;
(c) creep of concrete;
(d) flexural, shear, torsional, bursting or extensional stresses.

Shrinkage and temperature cracks may develop in a structural member because of the shape and/or where it is restrained against movement at its boundaries. In an adequately reinforced member, shrinkage cracks are seldom structurally dangerous but need to be repaired if in excess of hair-crack size in order to prevent the penetration to steel reinforcement of moisture with chlorides in solution. However, where the cracks run transversely to the reinforcement, corrosion is unlikely to take place (see Section 6.2.2). This also applies to flexural cracks. Where cracks run parallel to reinforcement the cause should be established, as it may indicate an advanced state of corrosion of the reinforcement. Furthermore, if such cracks have not been caused by the bursting effect of the corrosion products, they may be due to structural stresses, in which case they could result in corrosion of the reinforcement. Where tensile cracks have appeared because of overloading, differential settlement or under-design, it may be necessary to strengthen the member concerned. Various methods have been developed to increase the flexural shear or tensile strength as follows:

(a) Bonding mild steel grit blasted plates to the relevant surface of the concrete member with epoxy resin or other suitable adhesive substance. For corrosion protection the plates should be coated with an epoxy paint system.
    Small screws should be used to ensure that the plate is kept firmly in position until the epoxy resin has set (see Fig. 6.3). The epoxy layer thickness should be of the order of 1 mm.
(b) Adding reinforced concrete thickening, cast *in situ*, keyed into the parent member using concrete keys, steel dowels or friction due to transverse post-stressing, applied separately or in combination.
(c) Using external post-tensioned cables or rods suitably anchored and so positioned to counteract excessive tensile stresses. Figure 6.4 illustrates the case of a concrete box-girder that was strengthened to resist torsional and shear stresses. The longitudinal reinforcement in this case was adequate. Figure 6.5 shows the U-shaped stainless steel sheathing that was subsequently grout-filled to protect the cables against corrosion.

6.5.4.3 The Repair of Damage due to Corrosion of Steel Reinforcement

Before any work on repairing corrosion damage is commenced, it is essential to establish the root causes, which should if possible be eliminated or a method of prevention conceived to avoid a continuation of the corrosive process. Methods commonly employed vary from waterproofing the concrete by means of a range of suitable applications to the exposed surfaces of the concrete, to cathodic protection. Chloride contamination due to the use of de-icing salts on the deck can be prevented by sealing all affected surfaces. Various waterproofing applications have

Figure 6.3
Fixing steel plates epoxy-bonded to concrete soffit of a box-girder bridge (courtesy of S Roux)

been used on decks and it would appear that impervious membranes are the most effective, provided that they are bonded to the deck surface. Experts should be consulted in the selection and application of the preferred method.

It remains essential to ensure that all steel reinforcement has adequate cover and that the bonding and density of added concrete complies with the minimum standard to limit moisture penetration and the process of carbonation. It must be borne in mind that localized repair of a concrete structure does not guarantee that the corrosion will not subsequently occur in adjacent contaminated and carbonated parts. Studies in North America[6.50,6.51] have assessed the ability of patch repair systems, overlays, membranes and sealers to stop on-going corrosion of reinforced concrete bridge decks and conclude that none will stop corrosion once the contamination by chlorides has occurred. These studies have resulted in a Federal Highways Administration (FHWA) formal memorandum which concludes that 'the only rehabilitation technique that has proven to stop corrosion in salt contaminated bridge decks regardless of the chloride content of concrete is cathodic protection'. Cathodic protection[6.52] is achieved by causing a low-voltage direct current to flow from an anode system, placed on or in the surface of the concrete, through the concrete and to discharge on the reinforcing steel. This cathodic protection current opposes the current associated with the corrosion

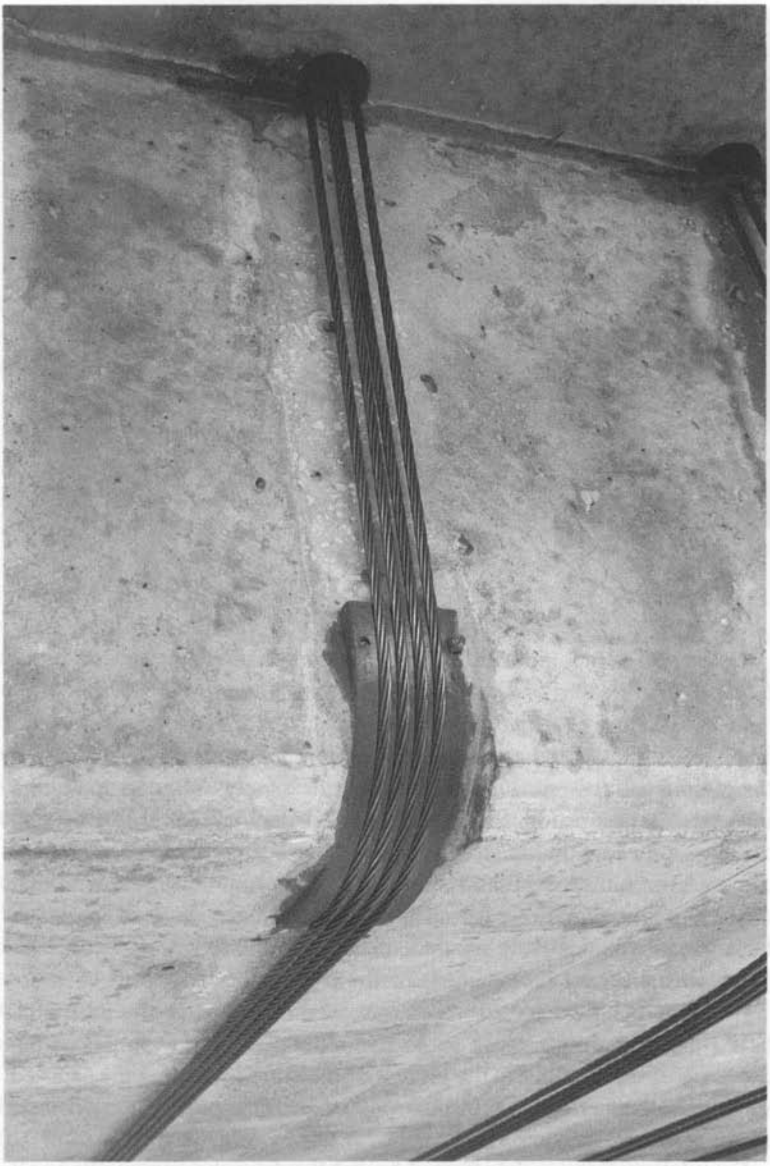


Figure 6.4 Post-tensioned cables to provide additional reinforcement against shear and torsion in a concrete box-girder bridge (courtesy of S Roux)

process occurring on the steel, thus balancing or nullifying the corrosion current, and thereby preventing corrosion. There are several cathodic protection systems,[6.51] e.g. conductive overlay systems, slotted anode systems and distributive mesh anodes. Patented conductive coatings have been developed specifically for bridge structures.[6.51]

Figure 6.5
Stainless steel sheathing to cables shown in Fig 6.4 grout-filled to protect cables against corrosion (courtesy of S Roux)

6.5.4.4 The Repair of Damage due to the Disruptive Effect of Alkali–Aggregate Reaction

The main effect of the expansive disruption due to AAR is to decrease the effective elastic modulus of the concrete as well as creep due to the internal reaction products and the crack formation. Tests by Blight *et al*. have indicated that in structures that have suffered a limited amount of deterioration the decrease in strength is not commensurate with that of the modulus.[6.14] However, if the deterioration is allowed to go too far, especially in relatively small members, it would be extremely difficult to repair except by replacement of the damaged parts.

Once the phenomenon has been observed in a concrete member, the best solution would appear to be the reduction or prevention of the exposure of the member to the ingress of moisture by sealing the exposed surfaces with an effective protective coating or by improving the drainage of the structure.

Foundations are rarely affected by AAR where the surrounding soil remains damp, but in those known cases where seasonal drying has apparently occurred the deterioration has been confined mainly to the upper edges of the base, which can be repaired by removal of the defective concrete with cutting tools in order to replace those portions with new concrete. This should be done in such a manner that the additional concrete effectively protects the base concrete and preferably becomes an integral part of the foundation structure. Various methods of keying the new concrete to the old have been proved, such as shaping the parent concrete to provide keys with or without steel dowels bonded into drilled holes. To control cracking of the new concrete due to differential shrinkage, adequate reinforcement should be placed parallel to the sides of the base, lapped round corners where necessary and held together by transverse binders.

Localized deterioration of pier capping beams have been caused by rainwater seeping through expansion joints in the deck (see Fig. 6.1). The capping beam after repair should be protected by sealing the joints and making provision for diverting any leakage by installing gutters and sheet metal protective cappings. In such cases improved drainage and a surface repair of the concrete may be adequate to maintain the structure in a satisfactory serviceable state.

According to Blight *et al.*[6.14] there are many possible ways of carrying out such a repair. The most drastic would be to chip away the affected concrete and replace it by a sprayed mortar or shotcrete. A less drastic, but more laborious, approach would be to inject the cracks with a suitable and compatible resin or some other adhesive substance, thus sealing out moisture and strengthening the deteriorated surface concrete. Cured resins have coefficients of thermal expansion as much as 50 times that of concrete. This movement has to be accommodated by creep in the resin.

In other cases, AAR attack may have affected the integrity of the structure and means may have to be adopted to restore this. Post-stressing is an obvious and convenient means of repair, although the increased creep to which AAR-affected concrete is subjected is disadvantageous.

6.5.4.5 The Repair of Concrete Subjected to Chemical Attack

Where chemical attack has occurred on concrete members of a bridge it is necessary as a first action to establish the source and nature of the suspected chemicals. Samples of affected and sound concrete should be taken and subjected to comparative examination and analysis. The analysis, as a general rule, should include at least:

(a) concentration of suspected chemical versus depth of concrete;
(b) detection of adverse reaction products to determine the character of the damage and likelihood of further damage.[6.49]

An expert opinion on the interpretation of the analysis is essential. Those parts of the concrete that have deteriorated to an unacceptable extent and/or have been penetrated should be cut out using suitable hand- or power-driven tools and taking great care not to damage any reinforcement, and the concrete and reinforcement should be repaired as described in Section 6.5.4.2.

6.5.4.6 *The Underpinning of Foundations*

Where a base has settled or heaved resulting in excessive differential deflections and/or regular observations indicate that the rate of settlement or heave has not decreased significantly, it may be necessary to underpin the base or provide additional support to the structure. It is essential that geotechnical experts establish the nature and causes of the movement.

The settlement of bases on granular material is usually a short-term phenomenon except where vibrations have a longer-term accumulative effect or poorly compacted sands are subsequently inundated and further settlement may take place. Settlement or heave on clay materials can be long-term and reversible. Structures supported on clay materials should therefore preferably be founded at a depth near or below the level of permanently saturated clay. Where the water-table is deep-seated and the clay of an expansive nature, under-reamed piles may be founded at an adequate depth to be determined by geotechnical experts and the piles and pile cap kept structurally isolated from the soil above the founding level of the piles to prevent the heaving effect of the upper layers being transferred to the structure. For spread footings, various underpinning methods may be applied. The size of the base may be increased at the original founding level by keying the surrounding additional concrete to the original base. Suitable reinforcing steel (including prestressing if necessary) should be provided to resist the torsional, bending and shear forces. Where feasible the base may be partially undermined and under pinned in stages. Alternatively piles may be used to augment the base. The piles are positioned around the base and the base enlarged (as explained above) to form a combined base and pile cap. Care should be taken to ascertain what the subsequent relative settlements are likely to be so as to ensure that the piles are not overloaded by excessive load transfer. Where considerable settlement has caused adverse stressing or cracking of the substructures or superstructure, it may be necessary to reinstate the structure to its original levels. Where the superstructure is supported on replaceable bearings, this can readily be done using hydraulic jacks and building up the bearing support surface to the correct level. Where there is structural continuity between the superstructure and sub-structure the operation is more difficult, as the column and/or base must be raised and underpinned at the correct level.

6.5.4.7 *The Replacement of Bearings*

Provided that the extremes of movement and rotation have been correctly assessed, bearings of good-quality stainless steel sliding on Teflon should last almost indefinitely, but if the limits are exceeded the best of bearings may be damaged (see Chapter 5, Section 5.8.2 and Figs 5.53–5.58). This applies especially to elastomeric (Neoprene) pad bearings (see Chapter 5, Section 5.8.2.1), which rely on shear and compression

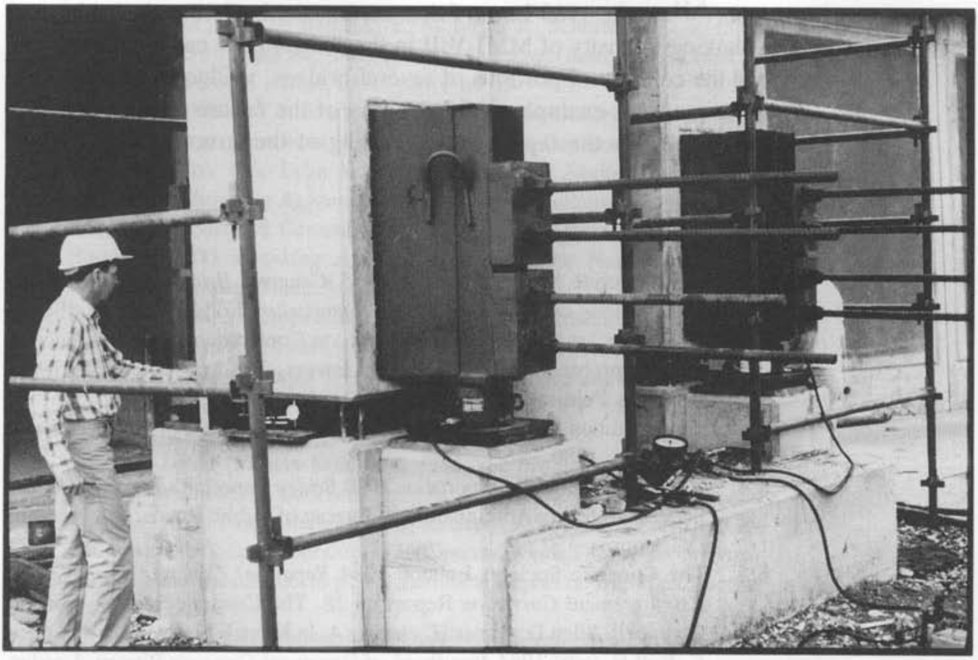


Figure 6.6 Steel brackets shear-keyed and post-tensioned to a column for the purpose of raising columns by jacking in order to insert bearings (courtesy of S Roux)

deformation of the pads to absorb the horizontal and rotational movements. Therefore, so long as provision in the design has been made, bearing pads can quite readily be replaced by using flatjacks between the pier head and the slab or beam soffit, or otherwise by using temporary steel propping or brackets (Fig. 6.6).

Such bearings should be replaced if they show obvious signs of deterioration such as excessive bulging or splitting of the edges of the Neoprene pads. Steel bearings may become clogged by extraneous material and should be kept clean. The Freyssinet-type bearings (see Figs 5.57 and 5.58) should be monitored for cracks or spalling of the concrete necks. Although this type of bearing has proved itself in extreme conditions, replacement should be considered if spalling becomes evident. It is important that bridges be constructed so that their bearings can be easily replaced.

6.5.4.8 *Severely Damaged Bridges*

The rehabilitation of bridges severely damaged, e.g. by ship collision, can be a major operation requiring specialized knowledge and experience. There have been several examples in recent years, such as the Great Beit Bridge disaster in Denmark, reported on in the *IABSE Proceedings* P-31/8, which deals with the investigation of ship collision problems. Another type of damage which is comparable is that due to seismic action as experienced in many parts of the world over the years. A recent report[6.53] on the 1989 Loma Prieta earthquake in California, which had a shaking intensity of MMI VIII in the central zone causing damage to and the collapse of portions of several bridges, viaducts and freeways, is an excellent example of the analysis of the failure mechanisms and procedures for the repair and upgrading of the structures.

References

6.1 The Concrete Society, London 1985 *Concrete Bridges, Investigation, Maintenance and Repair — Proc. Symposium* London 25 Sept 1985

6.2 Papers for 1st Ann. Conf. and Exibn on Construction Repair, Concrete Repair, problems, questions and answers, London, 16 March 1988, Palladian Publications 82 pp

6.3 The Institution of Structural Engineers, London 1980 *Appraisal of Existing Structures* 1980

6.4 US Department of Transportation 1970 *Bridge Inspector's Training Manual* Federal Highway Administration, Bureau of Public Roads, Washington, DC, approx 350 pp

6.5 The Concrete Society, London 1984 *Repair of Concrete Damaged by Reinforcement Corrosion* Report no 26, The Concrete Society, London

6.6 Campbell-Allen D, Roper H, chapter 4. In Kong F K, Evans R H, Cohen E, Roll F, (eds) 1984 *Handbook of Structural Concrete* Pitman, London

6.7 Neville A M 1963 *Properties of Concrete* Pitman, London, p 296

6.8 American Association of State Highway and Transportation Officials, Highway sub-committee on Bridges and Structures 1982 Chairman J Friedenrich Washington, DC 1983 *Manual for Maintenance Inspection of Bridges*

6.9 Stanton T E 1940 The expansion of concrete through reaction between cement and aggregate *proc. Am. Soc. Civil Eng.* **66**

6.10 Hansen W C 1958/1959 Expansion and cracking studied in relation to aggregate and the magnesium and alkali content of cement. *Proc. Am. Concr. Inst.* **55**

6.11 Davis D E 1981 Keynote address. In *Proc. Fifth Int. Conf. Alkali–Aggregate Reaction in Concrete* Cape Town, South Africa 1981

6.12 Skalny J, Klemm W A 1981 Alkalis in clinker-origin, chemistry, effects. In *Proc. Fifth Int. Conf. Alkali–Aggregate Reaction in Concrete* Cape Town, South Africa 1981, paper no 1

6.13 Svendsen J Alkali Reduction in cement kilns. *Proc. Fifth Int. Conf. Alkali–Aggregate Reaction in Concrete* Cape Town, South Africa 1981, paper no 2

6.14 Blight G E, Alexander M G, Schutte W K, Ralph T K The repair of reinforced concrete structures affected by alkali–aggregate reaction. In *International Concrete Symposium*, Concrete Society of Southern Africa, South Africa, May 1984

6.15 Oberholster R E 1981 Alkali–aggregate reaction in South Africa — a review. In *Proc. Fifth Int. Conf. Alkali–Aggregate Reaction in Concrete* Cape Town, South Africa 1981

6.16 Discussion of paper by Blight G E, McIver J R, Schutte W K, Rimmer R 1981. In *Proc. Fifth Int. Conf. Alkali–Aggregate Reaction in Concrete* Cape Town, South Africa 1981, paper no 15
6.17 Fulton F S, Webb T L Shrinking aggregates. In *Concrete Technology, A South African Handbook* 1977 Cape Portland Cement Institute p 150
6.18 Roper H, Cox J E, Erlin B 1959 *Petrographic Studies on Concrete Containing Shrinking Aggregate* Research Dept. Bulletin 172, Res. Dev. Lab. of the Portland Cement Ass., Illinois, USA
6.19 Roper H 1900 *Shrinking Aggregates in Concrete* National Building Research Institute, South Africa
6.20 Davis S G 1980 *The Inspection of Bridges* National Institute for Transport and Road Research, CSIR, Pretoria, Technical Report RSL/6/79
6.21 Frangopol D M, Curley J P 1987 Effects of damage and redundancy on structural reliability *J. Struct. Eng.* **113** (7): ASCE paper no 21677 pp 1533–1549
6.22 Frearson J P H (Messrs Sandberg) 1985 The problem — identification and interpretation. In *Concrete Bridges, Investigation, Maintenance and Repair — Proc. Symposium* London, 25 Sept 1985, The Concrete Society
6.23 The Concrete Society, London 1976 *Concrete Core Testing for Strength* (Part 4, other uses for cores p 32), Technical Report no 11, The Concrete Society, London
6.24 ASTM 1983 Standard recommended practice for petrographic examination of hardened concrete. *ASTM Standard C856 — 1983*
6.25 Cement and Concrete Association 1988 *Diagnosis of AAR in Concrete* Report of Working Party, Cement and Concrete Association
6.26 British Standards Institution 1971 Methods of testing concrete *BS 1881: Part 6: 1971*
6.27 British Standards Institution Methods of testing hardened concrete for other than strength *BS 1881: Part 5: 1970*
6.28 Levitt M Jul 1985 The ISAT for limit state design for durability. *Concrete* **19**(7) p 29
6.29 Levitt M 1984 *In situ* permeability of Concrete. *Symposium Developments in Testing Concrete for Durability* 26 Sept 1984, The Concrete Society
6.30 British Standards Institution 1983 Method for determination of the compressive strength of concrete cores. *BS 1881: Part 120: 1983*
6.31 British Standards Institution 1981 Guide to assessment of concrete strength in existing structures. *BS 6089: 1981*
6.32 British Standards Institution 1983 Methods for determination of density of hardened concrete. *BS 1881: Part 114: 1983*
6.33 Building Research Station 1981 *Carbonation of Concrete made with Dense Natural Aggregates* BRE Information paper IP 6/81
6.34 Building Research Station 1977 *Simplified Method for the Detection and Determination of Chlorides in Hardened Concrete* BRE Information Sheet IS 12/77
6.35 Building Research Station 1977 *Determination of Chloride and Cement Content in Hardened Portland Cement Concrete* BRE Information Sheet IS 13/77
6.36 ASTM 1982 Microscopical determination of void content and parameters

of the air-void system in hardened concrete *ATM Standard C457 — 1982*

6.37 British Standards Institution 1983 Method for determination of static modulus of elasticity in compression *BS 1881: Part 121: 1983*

6.38 Institution of Civil Engineers, London *Symposium on Non-destructive Testing of Concrete and Timber* 1970

6.39 British Standards Institution Testing of concrete *BS 1881: Part 200* series, to be published; 1984 *Part 201; Guide to the Use of Non-Destructive Methods of Test for Hardened Concrete* BS Draft for Public Comment, Document 84/10723 DC

6.40 Department of Transport 1983 *British Inspection Guide* HMSO, London

6.41 American Concrete Institute 1968 *Guide for Making a Condition Survey of Concrete in Service* ACI Committee 201. ACI Ref 201, IR-68 (ref 1979)

6.42 British Standards Institution Recommendations for non destructive methods of test for concrete *BS 4408: Part 1: 1969* Electromagnetic cover measuring devices. *Part 2: 1969* Strain gauges for concrete investigations. *Part 3: 1970* Gamma radiography of concrete. *Part 4: 1971* Surface hardness methods. *Part 5: 1974* Measurement of the velocity of ultrasonic pulses in concrete

6.43 Vassie P R 1980 *A Survey of Site Tests for Assessment of Corrosion in Reinforced Concrete* TRRL Lab. Report 953

6.44 ASTM 1980 Standard test method for half cell potentials of reinforcing steel in concrete *ASTM Standard C876 — 1980*

6.45 Wenner F 1915 A method of measuring earth resistivity *Bull. US Bureau of Standards, Science Paper* 12[3], 1915 pp 469–78

6.46 Millard S G 1991 Reinforced concrete resistivity measurement techniques *Proc. Inst. Civil Engr.* **91**(2) p 71–88

6.47 Woodward R J Jul 1983 Cracks in a concrete bridge. *Concrete* **17**(7) p 40–45

6.48 Keiller A P Feb 1985 Assessing the strength of *in situ* concrete. *Concrete Int.* Vol 7 No 2 p 15–21

6.49 Janney J R Maintenance, repair and demolition of concrete structures. In Kong F K, Evans R H, Cohen E, Roll F (eds) 1984 *Handbook of Structural Concrete* Pitman, London

6.50 Hay R E, Virmani Y P 1985 North American experience in concrete bridge deterioration and maintenance. In *Concrete Bridges, Investigation, Maintenance and Repair — Proc. Symposium* London, 25 Sept 1985, The Concrete Society

6.51 Wyatt B 1988 Cathodic protection of reinforced concrete — longterm repair strategy. 1st Ann. Conf. and Exibn on Construction Repair, London 1988 Palladian Publications p 43

6.52 The Institution of Structural Engineers, London 1988 *Cathodic protection of concrete structures — the way ahead. Papers presented at a seminar organized by the Inst. Str. Engrs in association with the corrosion control Engineering Joint Venture* 7 July 1988, London

6.53 Housner G W (chairman) *Report by the Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake* State of California, Office of Planning and Research May 1980

6.54 The Institution of Structural Engineers, London 1980 *Appraisal of Existing Structures* 253 pp

7 The Lessons Learnt from Failure of Bridges or Bridge Components

7.1 General

As described and discussed in Chapter 1, Section 1.2, the long history of failures extending into modern times and including long-span bridges[7.1,7.2] has taught the bridge building fraternity expensive but invaluable lessons and led to greater efforts in the form of both experimental and theoretical work to understand the forces that bridges are subjected to and their response thereto.[7.3–7.6] In time, this experience has culminated in a new and more sound philosophical approach to design and construction practice, as discussed in Chapter 3. Owing to the fact that this new approach has to some extent been incorporated in modern codes of practice, the probability of failures has been reduced. These codes cannot guard against all errors, however, especially those due to inexperience or negligence such as gross and conceptual errors and also accidents. In spite of modern computerized methods of design and quality assurance and control procedures in construction, failures — although less frequent — still occur, especially where new ground is broken. The greatest cause for concern, however, is the deterioration of bridges, as discussed in Chapter 6.

The main lessons that can be learnt from this history of failures and the persistence of a range of failures, from minor deterioration to catastrophic collapses, in spite of the amassing of so much experience and knowledge of the properties of materials and methods of analysis, are simply that humans err, that practical and economical constraints impose a need for marginal design which implies that risks have to be taken, and that there can never be absolute guarantees of safety (see Chapter 3, Section 3.2). Mankind in general seems mostly to have accepted such situations, e.g. motor-car and aircraft accidents, as unavoidable in everyday life.

7.2 The Human Factor

With reference to Chapter 3, and especially to the nature of the total design and construction process and the uniqueness of most bridges, it is clear that the manner and degree of human participation is such that the effects of negligence or ignorance and the consequential risk of gross errors is always real. This is borne out by many recent events. Studies are being done of the statistical significance of human error in design and the influence of systematic checking procedures.[7.7]

To reduce this risk and the underlying causes described in Chapter 3, Section 3.2.4, requires good organization with good management and effective lines of communication at all levels and times. Objectives and goals should be clearly defined and strategies and procedures planned accordingly.

This also implies good interaction between the various stages of activity with regular feedback for checking or comparison, review and confirmation, i.e. good teamwork. The events leading up to errors usually follow various patterns because of the peculiar attributes of individuals. These may be recognizable so that steps can be taken to reduce their occurrence.

In this context the selection of the leader of a design team is critically important. It is a fact that designers have various personality attributes and may be prone to certain types of oversight or error, usually because of a lack of an in-depth understanding of certain behavioural characteristics of structures. Such problems usually relate to secondary effects which may be magnified under certain conditions and may cause serious problems if disregarded in the mathematical modelling for the structural analysis. There have been several recent examples of such failures, especially during construction.

The accelerating development of computerized methods such as knowledge-based expert systems will open up many new opportunities and will greatly increase the efficiency and accuracy of analysis, especially of complex structures as well as the assessment of structural reliability. However, there will always be an inherent risk of human error due to the lack of complete understanding of users of the capabilities and limitations of specific programs. In some ways computerization has removed the need for the designer to fully understand some of the intricacies of structural behaviour and analysis which is a threat; on the other hand it has opened up vast opportunities for increasing such understanding and removing risks that have been inherent in design practice in the past.

References

7.1 Podolny W Jr, Scalzi J B 1976 *Construction and Design of Cable-Stayed Bridges* Wiley, New York, 506 pp

7.2 Report of the Royal Commission into the failure of West Gate Bridge, Government Printer, Melbourne, Australia, 1971, 143 pp

7.3 The modern design of wind-sensitive structures, *Proc. Seminar of Institution of Civil Engineers* London, 18 June 1970. Construction Industry Research and Information Association, London, 1971, 139 pp

7.4 Hay J S 1980 *An Introduction to Some Current Theories on the Aerodynamic Behaviour of Bridges* Transport and Road Research Laboratory, Dept of Environment, Dept of Transport, London, supplementary report 542, 50 pp

7.5 Simiu E, Scanlan R H 1978 *Wind Effects on Structures: an Introduction to Wind Engineering* Wiley, New York, chapter 6

7.6 Biétry J, Sacré C, Simui E 1978 Mean wind profiles and change of terrain roughness *J. Struct. Div. Am. Soc. Civ. Eng.* **104** (10): 1585–93

7.7 Melchers R E 1987 Human errors, human intervention and structural safety predictions *IABSE Proc.* P-119/87 pp 177–190

Index

Such enormous advances have been made in the design, construction and maintenance of concrete bridges in recent years that a review of the philosophy behind these disciplines is needed. *Concrete Bridges: Design and Construction* achieves this and begins by introducing the reader to the historical concepts of the development of the art of bridge building. Throughout the following chapters, the importance of aesthetics and the protection of the natural environment are emphasized. The author deals effectively with the subjects of design criteria for bridges and design philosophy, and explains the nature of innovation, and the concepts of risk, reliability and utility. A summary of suitable analytical procedures based on a range of mathematical modelling techniques and computer applications is included. Design and construction practices are described, and the importance of field inspection and maintenance emphasized. Case studies drawn from worldwide sources give credence to the international perspective of this unique reference work.

Throughout the text the author emphasizes the importance of costs, benefits and damages that society, the client and the natural environment may incur, and he shows the best way to maximize the efficiency of bridge construction.

The concluding chapters enumerate the causes and nature of deterioration of concrete bridges, as well as the methods of assessing damage, with explanations on effective repair techniques.

Practising civil and structural engineers will find this publication an ideal reference work on the current state of the art and science of design, construction and maintenance of concrete bridges. Students studying engineering design will find it a useful supplementary text on design philosophy.

A C Liebenberg is the Executive Chairman of Liebenberg & Stander, consulting civil and structural engineers practising in South Africa. He has been involved in the design of numerous engineering and building projects, mainly in South Africa, and in 1983 received the South African Association of Scientific and Technical Societies gold medal award for his achievements in engineering. In 1990 an award for Excellence of Service to the engineering profession was bestowed on him from the South African Society of Professional Engineers.

Longman's CONCRETE DESIGN & CONSTRUCTION SERIES comprises works on subjects of topical interest in the broad field of concrete technology. Written or edited by international authorities, the books will appeal to practising civil engineers, consultants, academics, students and others involved in this important area.

Cover photograph: Grant Smith/New Civil Engineer

ISBN 0-582-04281-X

Longman Scientific & Technical

Copublished in the United States with John Wiley & Sons, Inc., New York
ISBN 0470-21865-7 (USA only)