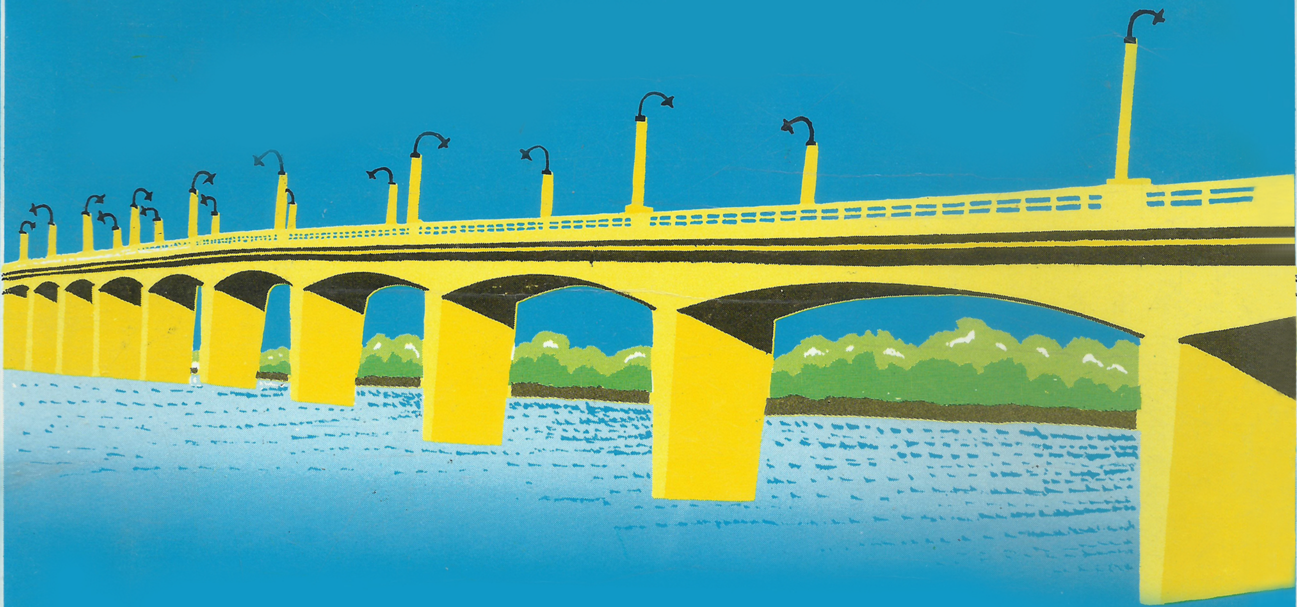
DESIGN AND CONSTRUCTION OF HIGHWAY BRIDGES

K.S.RAKSHIT



Design and Construction of HIGHWAY BRIDGES

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To my
Grand-son Soumitra
and
Grand-daughter Moumita

THE AUTHOR

The author is a retired Chief Engineer of the Public Works Department of the Government of West Bengal and was associated with the design and construction of various types of Bridges throughout his service career in various capacities starting from the Assistant Engineer upto the rank of Chief Engineer. In addition, he devoted considerable time in studying practical problems and published as many as 30 (thirty) technical papers dealing with theoretical and practical aspects of highways and highway bridges based on his long experience and field studies in the science of bridge engineering.

The author is a Life Member of the Indian Roads Congress and Life Fellow of the Institution of Engineers (India). The author acted as a member of the Bridges Committee as well as various other Committees of the Indian Roads Congress in connection with the preparation of the Standard Specification and Code of Practice for Road Bridges. The author was also a member of few code making committees of the Bureau of Indian Standards (ISI).

Due to his expert knowledge in bridge and structural engineering, the author was appointed both by the Ministry of Surface Transport (Roads Wing), Government of India, and the Government of West Bengal as a member of several technical committees to investigate into the causes of failure of some bridges and other structures.

THE BOOK

The book "***Design and Construction of Highway Bridges***" has been written for the benefit of both the students of bridge engineering as well as for the practising engineers. Keeping their needs in view, the Author has included in 25 (twenty-five) chapters, all aspects of bridge engineering starting from site selection, field investigations etc. and ending in construction and maintenance of bridges and bridge architecture.

To make the subject more clear and useful the book contains nearly 300 figures and sketches. Many illustrative examples are given in the book with a view to explaining the design aspects in terms of codal provisions and sound engineering practices.

Few special features which are the outcome of the Author's previous publications and long experience in bridge engineering such as load distribution in bridge decks, bearings & expansion joints, wearing course, river training & protective works etc. have enriched the contents of the book.

The book would be immensely useful to the students of bridge engineering as well as to the professional engineers associated with the design, construction and maintenance of bridges. The book would fully cater to the needs of these students, bridge designers and construction engineers.

*F*OREWORD

It is a pleasure to write a foreword for the book *"Design and Construction of Highway Bridges"* written by Shri K. S. Rakshit, B. Sc., B.E., F.I.E. The author, a retired Chief Engineer, PWD, West Bengal, is an eminent bridge designer and construction engineer. During his tenure of service in various capacities, he had been responsible for planning, design and construction of a number of highways and highway bridges, multi-storeyed buildings and sports complex. He has so far published 30 (thirty) technical papers some of which have been highly appreciated by the bridge designers including a foreign engineer. He was a member of the various Committees of the Indian Roads Congress which dealt with Bridge Codes. He also acted as a member of two Technical Committees, one for investigating the distress noticed in some box bridges on National Highway and the other, for finding out the causes of toppling over of the P. S. C. girder of the Calcutta End Interchanges of the Second Hooghly Bridge.

The author has in his book comprehensively dealt with various aspects relating to survey, investigations, fixation of waterways, depth of foundations, design of guide bunds, design and construction of various types of bridges viz. RCC slab, girder and arch bridges, prestressed concrete bridges, steel bridges, composite bridges and long span bridges. All sorts of foundations and their protective works as well as bearings and expansion joints have also been thoroughly described in the book.

Chapter 6 of the book will be of special interest to the bridge designers, which contains the author's simplified method of load distribution in bridge decks based on Morice & Little's theory. This method is being used in many bridge design offices with advantage.

Various methods adopted for the construction of foundations, erection and launching of girders, construction of long span arch bridges, truss bridges, cable-stayed bridges and suspension bridges have been dealt with meticulously. Even the proper method of layout of wells and checking of tilts and shifts have been elaborately explained.

Extensive use of figures and provision of good number of illustrative examples in the book to explain the design and construction aspects have enriched the quality of the book appreciably.

This book will undoubtedly be of immense help to the students, bridge designers and construction engineers. No word of praise is sufficient for the efforts made by the author in bringing out such a very useful book. I congratulate the author for the commendable endeavour.

R. B. SEN
Retd. Engineer-in-Chief & Secretary, P. W. D.
AND
Housing Commissioner, Govt. of West Bengal.

ACKNOWLEDGEMENTS

A number of books, journals, literatures and codes of various authors and publishers have been consulted in writing the book. I gratefully acknowledge the contributions of these authors and publishers as mentioned under ***"References"*** at the end of the chapters where necessary assistance from their publications has been received for the book. A few number of figures and tables have been taken from some of the aforesaid publications. This has been separately acknowledged mentioning the source within parenthesis where such materials are utilised.

Although adequate care has been taken to acknowledge the contributions of all the authors and publishers, whose assistance has been taken in preparing the manuscript, there might be few omissions in this regard which are unintentional and may be excused.

K. S. Rakshit

PREFACE

A good book on bridge engineering is required to supply to the user of the book, whether a university student or a professional engineer, the necessary information relating to the design, construction and maintenance of bridges so that the user can get the basic knowledge of bridge-engineering step by step viz. the site survey and field investigations, selection of bridge site and estimation of the design discharge, bridge loading and the possible maximum scour etc. and ultimately becomes conversant with the design, construction and maintenance of various types of bridges normally used in practice.

The purpose of writing the present book dealing with the design, construction and maintenance of highway bridges is two fold viz. (a) to have more coverage on the subject embracing all aspects of highway bridge engineering and (b) to explain the principles involved in the bridge design followed by sketches and illustrative examples for better understanding of the applications of the theory, formulae and the codal provisions involved.

As regards the coverage, the book contains as many as 25 chapters embracing all aspects of highway bridges. No book on highway bridges has, perhaps, such a wide coverage. These 25 chapters present a comprehensive treatment of the subject of bridge engineering starting from the historical development of bridges and ending with the bridge architecture. The intermediate 23 chapters deal with the design, construction and maintenance aspects of highway bridges.

The reason for explaining the use of formulae and codal provisions by way of illustrative examples in the present book is that I felt the necessity while going through some good engineering books wherein such illustrative or worked out examples were absent. I realised that the theories explained in those books could have been better understood and utilised in practical works had the theories or the basic principles described in the book been explained by illustrative examples.

In addition to more coverage, some important chapters have been dealt with in more details in this book such as Chapter 8 (T-beam & Slab), Chapter 9 (Skew and Curved Bridges), Chapter 16 (PSC Bridges), Chapter 17 (Long Span Bridges), Chapter 18 (Temporary, Low Cost & Movable Bridges), Chapter 21 (Shallow & Deep Foundations), Chapter 22 (Bridge Bearings, Expansion Joints, and Wearing Course) and Chapter 24 (Construction, Erection and Maintenance of Bridges). Chapter 6 and 23, however, have some unique and special features respectively. Unique feature of Chapter 6 is that along with standard load distribution theories such as Courbon's and Morice & Little's load distribution theories, this chapter contains my simplified method of load distribution in bridge decks based on Morice & Little's theory. This simplified method was first published by me and was thereafter widely used by the bridge designers. Chapter 6 also deals with my method of finding out Morice & Little's distribution coefficients directly from Courbon's values within certain range of span width ratio.

The contents of Chapter 6 cannot be found in any book on bridge engineering. The subject matter of Chapter 23 viz. River Training and Protective Works for Bridges is rarely found in such great details in the books of bridge engineering.

Drawing is the language of engineers. Keeping this idea in view, the book has been profusely illustrated and contains 300 illustrations (sketches), i.e., three illustrations in every four pages on an average. To make the book more useful to the readers, the book also contains as many as 76 tables and 33 illustrative examples and designs. In addition, 17 curves of my simplified method of load distribution and 8 curves of Pigeaud's method of deck slab design have been incorporated in Appendix B and C respectively.

I acknowledge my indebtedness to Shri S. K. Ghosh, retired Chief Engineer, Public Works (Roads) Department, Government of West Bengal and a renowned bridge engineer, from whom many new aspects of bridge engineering were learnt while working under him as Executive Engineer of one important bridge design division. I also express my deep gratitude to Shri R. B. Sen, retired Engineer-in-Chief & Secretary, Public Works Department and Housing Commissioner, Government of West Bengal, for kindly writing the ***"Foreword"*** for the book. I am grateful to Shri N. C. De, retired Chief Engineer, P. W. D. for going through the manuscript copy of the book and giving valuable suggestions. I appreciate the co-operation received from Smt. Reba Rakshit (wife) and Smt. Suparna Rakshit (daughter-in-law) during the preparation of the book. Unless they patiently and ungrudgingly tolerated the inconveniences in day to day family affairs and thus kept me free and undisturbed in writing the book, the publishing of the book would not have been possible.

I am deeply indebted to M/s. New Central Book Agency, the publisher of this book, for bringing out the book with such good and beautiful get-up, presentation, printing and binding. They spared no pains in achieving their ultimate goal viz. publishing a book of good quality for which they have the reputation.

I hope that the book which is written with utmost devotion and hard labour to meet the needs of the students and the professional engineers will achieve the goal and be useful to the users. If this desire of mine is fulfilled, I shall consider that my hard labour has been fully repaid and my long cherished dream viz. publishing a good book on bridge engineering has come true.

In conclusion, I request the readers to bring to my notice such mistakes that might have crept in in spite of all cares. Suggestions for the improvement of the book are welcome from the readers and for this act of kindness I shall remain grateful to them.

K. S. Rakshit

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CHAPTER 1

HISTORY OF BRIDGES

1.1 DEFINITION OF A BRIDGE

1.1.1 A bridge is a structure which maintains the communications such as the road and railway traffic and other moving loads over an obstacle, namely, a channel, a road, a railway or a valley. The structure is termed as a ***"Bridge"*** when it carries road and railway traffic or a pipe line over a channel or a valley and an ***"Overbridge"*** when it carries the traffic or pipe line over a communication system like roads or railways. A ***"Viaduct"*** is also a bridge constructed over a busy locality to carry the vehicular traffic over the area keeping the activities of the area below the Viaduct uninterrupted.

1.2 HISTORY OF DEVELOPMENT OF BRIDGES

1.2.1 The history of the development of bridges is closely associated with the history of human civilization. The art of bridge building has, therefore, attracted the attention of the engineers and builders from the beginning of the civilization. It may be well presumed that the idea of building a bridge across an obstacle such as a channel or water course occurred in human mind by observing natural phenomenon such as a tree trunk fallen accidentally by a storm across a small water-course or a piece of stone in the form of an arch over a small opening caused by erosion of soil below or a bunch of creepers from one tree to the other used by monkeys. It may also be well imagined that in those old days, a man of intelligence and initiative was perhaps encouraged by the above mentioned natural phenomena and built bridges over a small water- course by placing a piece of log or by tieing a bunch of long creeper with the trees situated on either side of the water course. The above two illustrations indicate without any doubt that the former was the predecessor of girder type bridges and the latter was the fore-bearer of suspension bridges. Though the methods adopted in those days to cross the small water-course was of primitive nature, it cannot be denied that those were the beginning of the science of bridge building which has come to the present state of development through continuous search and constant effort for constructing longer and stronger bridges by using new technique and better bridge building materials.

Table 1.1 gives an account of the development of bridges from the ancient times to the modern age.

1.2.2 The earliest bridge on record was the bridge over the Nile built by Menes, the king of Egypt about 2650 B.C. Five centuries later, another bridge was built by Queen Semiramio of Babylon across the river Euphrates. A number of arch bridges were built by Mesopotamians, the Egyptians and the Chinese. The

TABLE 1.1 DEVELOPMENT OF VARIOUS BRIDGES FROM THE ANCIENT TIMES TO THE MODERN AGE

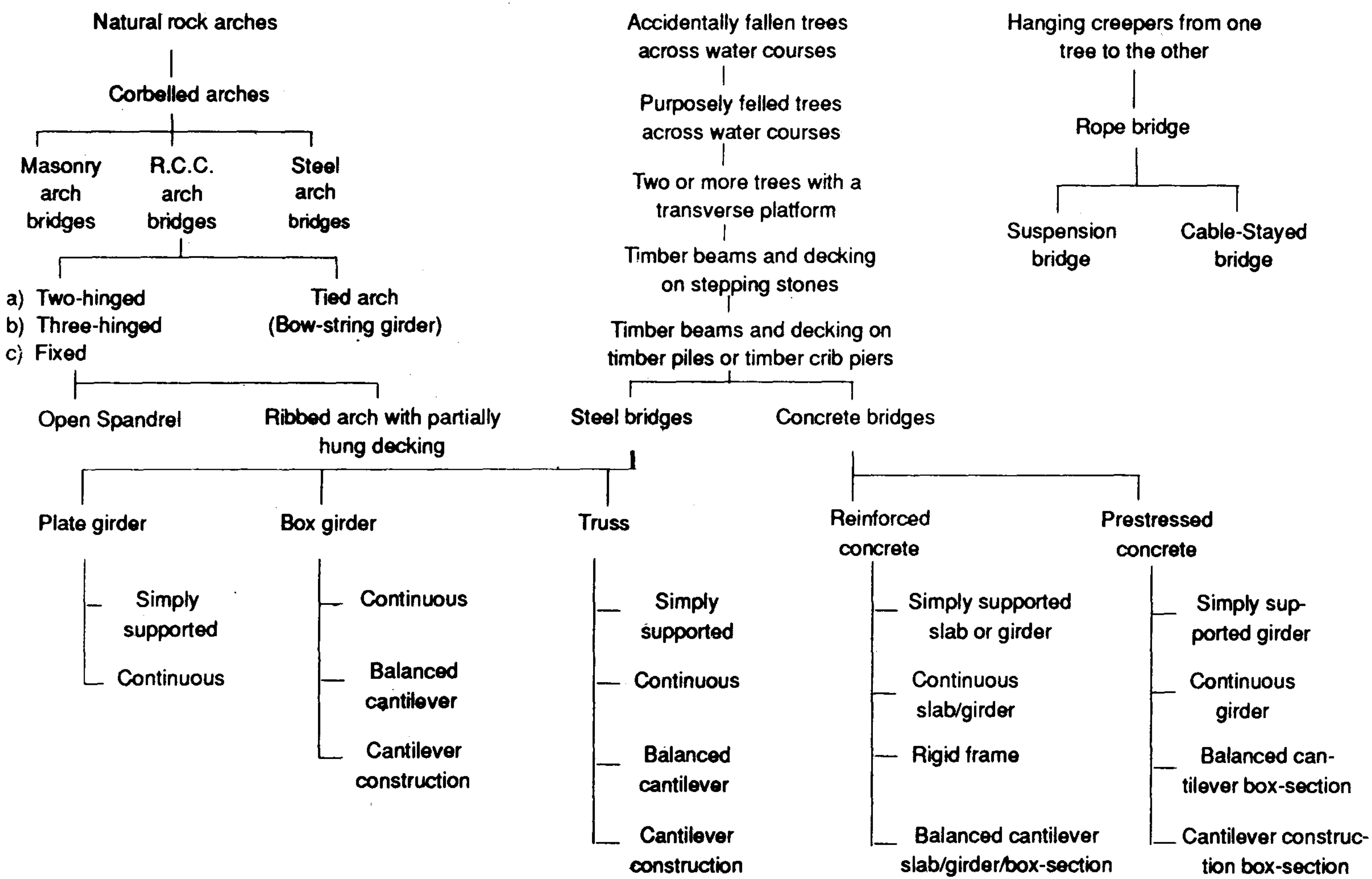


TABLE 1.2 LIST OF FEW BRIDGES FROM ANCIENT TIMES TO MODERN AGE

Sl. No.	Name of Bridge	Year of construction	Location	Type	Main span
1	Martorell	219 B.C.	Spain	Stone Arch	37 metres
2	Nera River	14 A.D.	Italy	Stone Arch	43 metres
3	Loyang	1059 A.D.	China	Stone girder & slab	11 metres
4	Trezzo	1377 A.D.	Italy	Stone Arch	77 metres
5	schuylkill Falls	1816 A.D.	Philadelphia	Suspension	124 metres
6	Wheeling	1849 A.D.	Ohio River	-do-	308 metres
7	Clifton	1869 A.D.	Niagra Falls	-do-	386 metres
8	Brooklyn	1883 A.D.	New York City	-do-	486 metres
9	Firth of Forth	1889 A.D.	Scotland	Cantilever	521 metres
10	Hell Gate	1917 A.D.	New York City	Arch	298 metres
11	Paducah	1929 A.D.	Ohio	Simple Truss	218 metres
12	Sydney Harbour	1932 A.D.	Sydney	Steel Arch	503 metres
13	Golden Gate	1937 A.D.	Sanfransisco	Suspension	1280 metres
14	Howrah	1943 A.D.	India	Cantilever	457 metres
15	Mackinac Straits	1957 A.D.	Mackinaw City, Michigan	Suspension	1158 metres
16	Gladesville	1964 A.D.	Sydney, Australia	Concrete Arch	305 metres
17	Astoria	1966 A.D.	Collumbia River	Continuous Arch	376 metres
18	Bonn - sud	1971 A.D.	Bonn, W. Germany	Continuous Box Girder	125-230-125 metres
19	Zorate - Brazo	1972 A.D.	Parana River, Argentina	Cable Stayed (Steel)	340 metres

Chao-chow bridge over Hsiaoho River was built by the Chinese around 600 A.D. about 300 km south of Peking (now Beijing). It was a single span of 37.4 metres long stone arch bridge. The Romans were known to be the best bridge builders between 200 B.C. and 260 A.D. and some of the masonry arch bridges built by them still exist. The arch bridge known as ***Pont-Du-Gard*** was built in France in 14 A.D. and due to good preservation and maintenance, this bridge is still in a fairly good condition. In ancient Rome, the Roman emperors adopted the title ***"Pontifex Maximus"*** meaning ***"Chief Bridge Builder"*** which indicates that the Romans attached great importance to the bridge construction. It was actually the Romans who first took up bridge building in a systematic manner. They knew the use of pozzolana and made good use of this in making masonry bridges. The Romans built large arches and viaducts but the weakest part of their bridges was their foundation as they did not have knowledge of river scour resulting in collapse and damage to most of the bridges built by them in course of time.

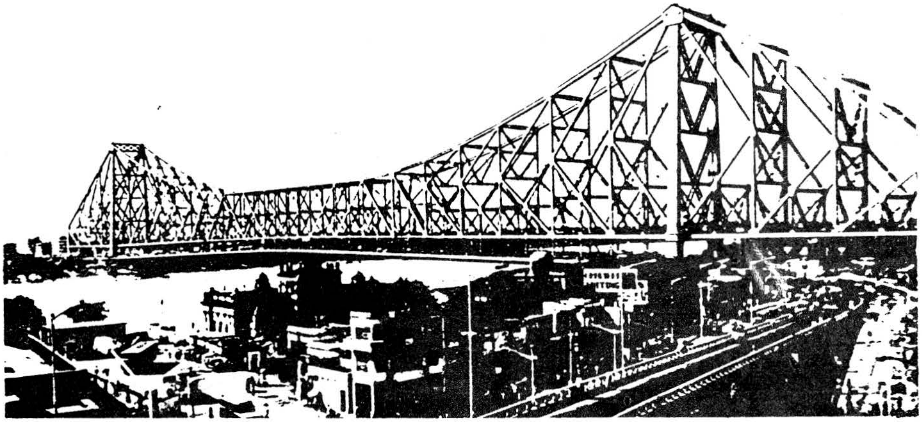
1.2.3 The bridges constructed during the middle ages which need special mention were the ***Thames Bridge*** at London and ***Ponte Vecchio*** over river Arno in Florence. The former bridge had been built in 1209 and was in use for more than six centuries while the latter was built in 1177. The special features of these bridges were that in addition to the bridge decks those bridges used to provide decorative and defensive towers, chapels, statues, shops and dwellings. The bridges of importance built in the 16th Century were the ***Santa Trinita Bridge*** over the Arno in Florence (1569) and the ***Rialto Bridge*** over the Grand Canal in Venice (1591).

1.2.4 The era of modern bridging activity started in 18th century when the use of iron was made by some of the famous bridge engineers of that time. Initially, cast iron was used for construction of a number of arch bridges but wrought iron gradually replaced the cast iron which in time was also replaced by steel when Bessemier Process of steel making was introduced. During this time, the design and construction of the bridges were tried to be based on scientific theories. The first treatise in bridge engineering was published in 1714 by Rubert Gautier, a French engineer. ***The Corps Des Ingenieurs Du Ponts Et Chaussees*** was founded in 1716 for the advancement of bridge construction. ***The Ecole De Ponts Et Chaussees,*** the first engineering school in the world, was established in Paris with Jean Perronet, the father of modern bridge building, as the first Director

1.2.5 During 18th-19th century, first iron bridge of 30.5 metres span was built in 1779 over the ***Severn*** in Coalbrookdale, England by Abraham Derby and John Wilkinson. The first steel bridge was the ***Eads Bridge*** constructed at St. Louis, Missouri in 1874. This bridge was a steel arch bridge of three spans of 153 m, 158 m and 153 m. With the introduction of steel as bridge building material in place of iron, the earlier form of trusses, such as Bollman, Fink, Howe, Pratt, Warren, Whipple etc. were replaced by more efficient forms such as Baltimore, K-truss, Parker etc. With the improvement of the quality of steel, there was rapid development in the construction of large span steel bridges, particularly the suspension as well as the large steel cantilevers. The British engineers, Telford, Stephenson etc. built many interesting bridges but they did not have adequate knowledge for safeguarding the bridges against sway and vibrations caused by high winds and movement of heavy dynamic loads. The result of this lack of knowledge was the collapse of many suspension bridges.

1.2.6 The world's first modern cantilever bridge was built in 1867 across the river ***Main*** at Hassfurt, Germany with a main span of 12[illegible] m. The world's largest span cantilever bridge is the ***Quebec Bridge*** built in 1917 over the St. Lawrence river, Canada. The main span of this bridge was 549 m. The first cantilever bridge built in India is the ***Howrah Bridge*** over the river Hooghly at Calcutta (1943). This bridge is the fourth longest cantilever bridge in the world having a main span of 457 metres. (Photograph 1)

1.2.7 In the 19th century due to manufacture of heavy load lifting equipments and high capacity compressors, pneumatic sinking of caissons in deep water became possible. As a result, construction of large spans in deep water could be taken up. The bridge over ***Schuylkill Falls*** with a main span of 124 metres was constructed in 1816 at Philadelphia. The ***Fribourg Bridge*** was built in Switzerland in 1834 with a span of 265 metres. The



Photograph 1
Howrah Bridge at Calcutta *(Highway Bridges)* [1]

Cincinnati Bridge over Ohio River was constructed in 1867 and the ***Brooklyn Bridge*** at New York City was built in 1883. The former bridge had a main span of 322 metres and the latter bridge had a main span of 486 metres.

1.2.8 The production of alloy steel and manufacture of cement and heavy construction equipments together with the advanced knowledge of the theory of structures and better understanding of the effect of dynamic forces such as wind etc. on the structures resulted in the construction of some famous arch bridges, cantilever bridges and suspension bridges (Table 1.3).

TABLE 1.3 : LIST OF SOME FAMOUR ARCH, CANTILEVER AND SUSPENSION BRIDGES

Sl. No.	Name	Date of completion	Type	Span (metres)	Capacity
1	Hell Gate	1916	Arch	298	4 Railway tracks
2	SydneyHarbour	1932	-do-	503	4 Inter urban tracks 6 Lanes roadway 2 Footway
3	Quebec	1918	Cantilever	549	2 Railway tracks 2 Footway
4	Howrah	1943	-do-	457	2 Tramway 6 Lanes roadway 2 Footway
5	George Washington	1931	Suspension	1067	8 Lanes roadway
6	Golden Gate	1937	-do-	1280	6 Lanes roadway 2 Footway
7	Verrazano Narrows	1966	-do-	1299	6 Lanes roadway

1.2.9 Reinforced concrete bridges gained popularity in the 20th century because of their versatility in construction and economy in cost and maintenance. In addition to their aforesaid advantages, reinforced concrete bridges can be cast in any convenient shapes and forms to meet architectural requirements as well as can utilise locally available materials such as stone chips, gravels, sand etc. and can be cast at site thereby eliminating carriage of heavy bridge components from fabrication workshop as required for steel bridges. It is for these reasons that reinforced concrete has practically replaced the use of steel as a chief building materials in the construction of small to medium span bridges specially highway bridges except in case of long span bridges where the steel with its improved quality is the only bridge building materials. Reinforced concrete is finding its place even in these bridges in bridge components like towers, decks etc.

1.2.10 One of the longest span reinforced concrete bridges is the ***Sando Bridge*** built in Sweden in 1943 with a span of 264 metres. Only about a decade ago, the new ***Sydney Harbour Bridge*** which is an R.C.C. arch bridge having a span of 305 metres had been constructed.

1.2.11 Manufacture of high strength concrete and prestressing the same by the use of high tensile steel wires further improved the construction of concrete bridges. These bridges known as prestressed concrete bridges, have some more advantages over the reinforced concrete bridges, namely, they are cheaper for medium span bridges, can be precast at the bed or at the approaches and lifted by derricks/cranes or launched by launching trusses and placed at their final position thus eliminating the use of costly staging. These bridges are ideally suitable in deep rivers where staging in deep water is difficult and launching of the girders in their respective locations by the use of launching trusses is the only answer. For comparatively larger spans, the use of launching trusses has now been avoided by using a new technique, namely, ***"Cantilever construction"***.

1.2.12 One of the early prestressed concrete bridges is the ***Marne Bridge*** built in France. The first prestressed concrete bridge built in India (Tamilnadu State) is the ***Palar Bridge.*** The long span prestressed concrete bridges in India have been constructed by the cantilever method of construction. Some of these bridges are ***Barak Bridge*** at Silchar, Assam with a central span of 122 m, ***Bassein Creek Bridge*** near Bombay with two central spans of 115 m each, ***Ganga Bridge*** at Patna with a number of central spans of 121-m each. The longest span prestressed concrete bridge built in India by cantilever construction method is the ***Lubha Bridge*** in Assam with a central span of 130 m.

1.2.13 Recent addition to the development of modern bridges is the cable-stayed bridges. The concept and practical application of the principle of cable stayed bridges were not new and a Venetian engineer named Verantius built a bridge of the type with several diagonal chain-stays as far back as 1600 A.D. The modern version of the cable-stayed bridges was first used in 1950 in Germany. Till then, a number of cable-stayed bridges have been built in many parts of the world. Some important cable-stayed bridges in various countries are illustrated in Table 1.4.

TABLE 1.4 : LIST OF SOME IMPORTANT CABLE-STAYED BRIDGES OF THE WORLD

Sl. No.	Name of Bridge	Location	Year	Span (Metres)
1	North Bridge at Dusseldorf	West Germany	1958	260
2	Maracaibo	Venezuela	1962	235
3	USK River	Great Britain	1964	152
4	Bridge near Leverkusen	West Germany	1965	280
5	Duisburg - Neuenkamp	West Germany	1970	350

TABLE 1.4 : LIST OF SOME IMPORTANT CABLE-STAYED BRIDGES OF THE WORLD [Contd.]				
Sl. No.	**Name of Bridge**	**Location**	**Year**	**Span (Metres)**
6	Bridge over the Rheine	Mannheim	1971	287
7	Danube	Czekoslovakia	1971	303
8	Erskine	Scotland	1971	305
9	Lower Yara	Australia	1972	336
10	Saint Nazaire	France	1974	400
11	Danube	Navi-sad	1981	351

1.2.14 One of the longest cable-stayed bridges is under construction over the Hooghly river at Calcutta. This bridge has a main span of 457 metres with two side spans of 183 metres thus making the bridge length equal to 823 metres. Fan-type cables are used from the towers to support the deck. The towers are composed of steel box sections. The deck system shall consist of three steel girders with stringers and cross-girders to support the reinforced concrete deck over them having properly designed shear-connectors for composite action under live loads.

TABLE 1.5 THE WORLD'S LONGEST SPAN BRIDGES OF VARIOUS TYPES					
Sl. No.	**Type of Bridge**	**Name of Bridge**	**Location**	**Year of construction**	**Span, metres**
1	Masonry Arch	Plauen	Germany	1903	90
2	Cantilever	Quebec	Canada	1917	549
3	Concrete Girder	Villeneuve	France	1939	78
4	Tubular Girder	Britannia	Menai Strait	1850	140
5	Continuous Plate Girder	Sava I	Belgrade, Yugoslavia	1956	261
6	Vertical Lift	Arthur Kill	Elizabeth, N. J.	1959	170
7	Concrete Arch	Gladesville	Sydney, Australia	1964	305
8	Cable Suspension	Verrazano Narrows	New York	1964	1298
9	Continuous Truss	Astoria	Collumbus River, Oregaon	1966	376
10	Prestressed Concrete Girder	Urato Bay	Japan	1974	230
11	Cable-stayed Concrete Girder	Tiel	Netherlands	1974	267
12	Simple Truss	J. J. Barry	Delaware River	1973	251
13	Continuous Girder	Niteroi	Brazil	1974	300
14	Cable-stayed — Steel Girder	Saint Nazaire	France	1974	400
15	Steel Arch	New River Gorge	West Virginia	1976	519

1.2.15 The bridge builder has always a desire to build new type of bridges, either new in concept or new in technique of construction or new in shape and form or new in the use of building materials. The bridge engineer has also a desire to build longer and longer bridges of the same type exceeding the previous span lengths. To him, this is a challenge which he must meet to show that the science of bridge building is continuously developing. The bridge builders have built various types of bridges depending upon the surroundings, navigational and other technical requirements, availability of materials, foundation condition etc. The world's longest span bridges of various types are illustrated in Table 1.5.

1.3 CONCLUSION

The evolution of bridges from the ancient times to the present age is a continuous process and is a result of human desire to use more and more improved methods and materials in order to build cheaper, finer and stronger bridges of longer spans and of lasting quality. The search for further improvement is not over and will never be over. The bridge designers and the bridge builders will continue their search and experimentations for building cheaper, stronger and aesthatically better bridges for all times to come.

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CHAPTER 2

SURVEY AND SELECTION OF BRIDGE SITE AND COLLECTION OF DESIGN DATA FOR BRIDGE PROJECTS

2.1 PRELIMINARY WORKS

2.1.1 Collection of General Data

This includes preparation and collection of the following maps :

(a) ***Index Map :*** An index map showing the proposed location of the bridge, the existing means of communication, the general topography of the area and the important towns and villages in the area. This map shall be drawn usually in 1 : 50,000.

(b) ***A Contour Plan*** of the Stream showing all the topographical features for a sufficient distance on either side of the site to give indication of the features which would influence the location and the design of the bridge and its approaches. All the probable bridge sites under consideration shall be indicated on the plan. The distances to be covered both U/S and D/S side of the bridge site shall be as follows :

i) 100 metres for a small bridge or culvert or when the catchment area is less than 3 Sq. Km (scale 1/ 1,000).

ii) 300 metres for less important bridges or when the catchment area varies from 3 to 15 Sq. Km. (scale 1/1000).

iii) 1500 metres for important bridges or when the catchment area is more than 15 Sq. Km (scale 1/5000).

2.1.2 Preliminary Survey

The objective of the preliminary survery is to study more than one alternative bridge sites. Usually the probable bridge sites are first located in topo sheets (either in scale 1 : 50,000 or in 1 : 250,000) and thereafter these sites are visited to collect certain preliminary data required for thorough examination of the alternative bridge sites from which the final site shall be selected.

2.1.3 Collection of Preliminary Data for Selection of Bridge Sites

The bridge site shall satisfy the following requirements and as such the preliminary data for the selection of alternative bridge sites shall be collected accordingly. If all the requirements can not be satisfied there may be some compromise for the less important ones.

i) The channel is well defined and narrow.

ii) The river course is stable and has high and stable banks.

iii) The river has large average depth compared to localised maximum depth to ensure uniform flow.

iv) The bridge site shall be far away from the confluence of large tributaries specially at the upstream so that the site remains beyond the disturbing influence of them.

v) Whether the river meanders and if so locate the nodal points of the river course which are not affected by the meandering. Such nodal points may be a possible bridge site (Fig. 2.1).

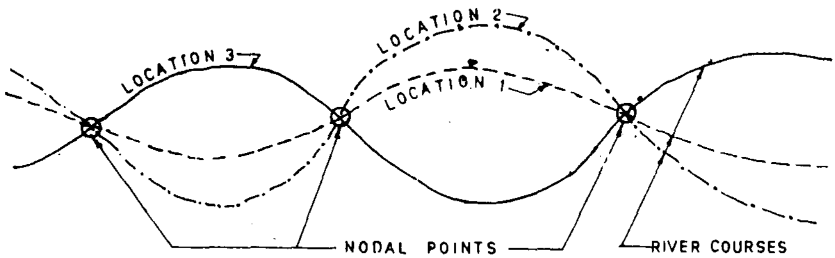


FIG. 2.1 NODAL POINTS OF RIVER COURSES

vi) The site shall have a straight approach road and a square crossing. Curves in the immediate approaches to the bridge shall be avoided. Skew crossing may be acceptable in unavoidable circumstance where curves in the immediate approaches to the bridge has to be accepted if square crossing is adopted (Fig. 2.2).

vii) The site is easily approachable from all sides and will give maximum service to the locality for which the bridge will be constructed.

viii) The proposed bridge will connect the road alignment, existing or proposed, with shortest approach roads.

ix) The site shall avoid curves at the immediate approach to the bridge. Curvature in the bridge proper shall also be avoided unless forced by site conditions. One such example is shown in Fig. 2.3 from author's personal experience in which the bridge was to connect the new road with an existing road running on one of the banks of the canal. A curved bridge was required to be constructed over the canal in order to avoid introduction of a S- curve in one of the approaches in addition to building a skew bridge. Square crossing as per alternative proposal No. 1 would have made the approach worse than alternative proposal No. 2 (Fig. 2.3-a). Curves may not perhaps be avoided in many bridges of the hill roads for shortage of space in the approaches.

x) The bridge site shall be such that there shall be no need for costly river training works.

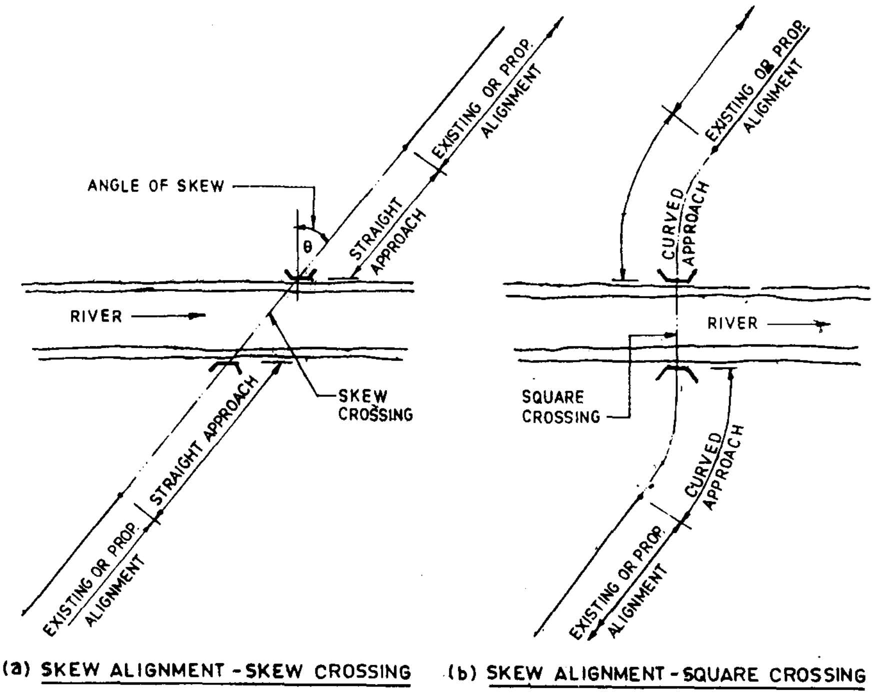


FIG. 2·2 SKEW AND SQUARE CROSSING

xi) The site shall be sound from geological consideration.

xii) Materials and labour required for the construction of the bridge shall be available as much as possible in the vicinity of the bridge site.

2.2 DETAILED RIVER SURVEY

2.2.1 On examination of the preliminary data collected for the selection of alternative bridge sites as stated in Art. 2.1.2 and 2.1.3 and in consideration of merits and demerits of all the alternative sites, the bridge site which satisfy most of the requirements for an ideal bridge site shall be finally selected. Once this is done, detailed river survey for this selected site shall be conducted on the items as outlined below:

a) A plane table survey plan of the stream showing therein all topographical features both upstream and downstream of the proposed site upto 200 to 500 metres on either side depending upon the size of the stream.

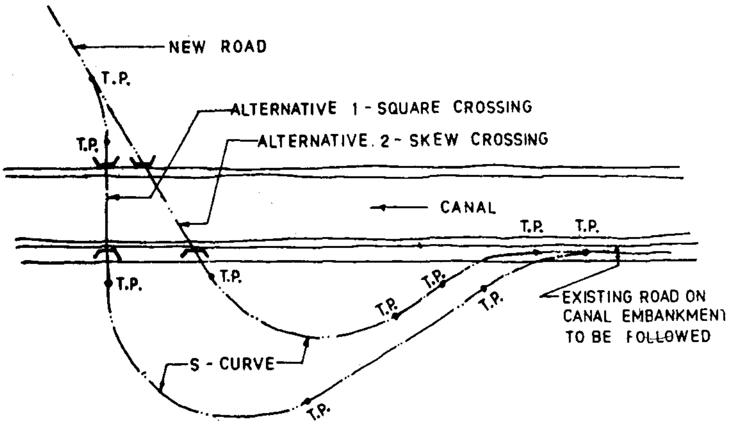
The following information shall be recorded in the drawing :

i) Name of the river and the road.

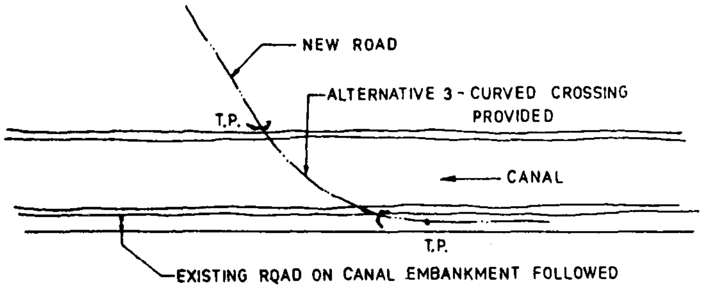
ii) Approximate outline of the banks.

iii) Direction of flow of water.

iv) Alignment of the existing or proposed approaches upto 500 m on either side and the angle of skew, if any.



(a) STRAIGHT BRIDGE (SQUARE OR SKEW CROSSING) WITH S - CURVE AT APPROACH



(b) CURVED BRIDGE

FIG. 2·3 ALTERNATIVE CROSSINGS

v) Name of the nearest town.

vi) Reference to the Bench Mark and its reduced level.

vii) The lines and identificatiron numbers of the cross- section and longitudinal section taken within the scope of the plan.

b) For the determination of the area of flow at H.F.L., Cross-Sections of the river, one along the proposed bridge site and few more both at upstream and downstream at 50 metre intervals shall be taken showing therein the information listed below :

i) The bed line upto the top of the banks and the ground line to a sufficient distance beyond the edge of the stream with levels at intervals sufficiently close to give a clear outline of the bed or ground.

ii) Low Water Level (L.W.L.)

iii) Ordinary Flood Level (O.F.L.)

iv) Highest Flood Level (H.F.L.) and the year in which it occured.

v) Maximum surface velocity at bridge site.

c) Longitudinal Section of the stream showing the lowest bed level for a distance of 500 metres both on upstream and downstream of the bridge site for the determination of the bed slope.

d) Nature of stream, namely -

i) Alluvial with erodable bed, or

ii) Quassi alluvial i.e., fixed bed but erodable banks, or

iii) Rigid with inerodable bed and banks.

This is requied for the fixation of waterway, scour, afflux etc. and whether,

iv) Perennial,

v) Seasonal, or

vi) Tidal

e) Nature of bed for the determination of ***"Rougosity Coefficient"*** (vide Chapter 3) namely,

i) Clean bed with straight banks

ii) Rifts or deep pool with or without some weeds or stones.

iii) Winding, some pools and shoals but clean with or without weeds or stones.

iv) Stoney sections with ineffective slopes.

v) Sluggish river reaches, weeds with deep pools.

vi) Very weedy reaches.

f) Nature of bed materials, namely,

i) Silt, or

ii) Sand (fine/medium/coarse), or

iii) Shingles/gravels/boulders.

This information is necessary for the determination of ***"Silt factor"*** (See Chapter 3).

2.3 ADDITIONAL INFORMATION TO BE COLLECTED

(a) ***Catchment Area and Run-off Data*** as listed below for the estimation of design discharge (See Chapter 3)

i) Catchment area — in hilly portion and in plain portion separately.

ii) Maximum recorded intensity and frequency of rainfall in the catchment.

iii) Rainfall in cm per year in the region.

iv) Length of catchment in Km.

v) Width of catchment in Km.

vi) Longitudinal slope of the catchment.

vii) Cross-slope of the catchment.

viii) The nature of catchment and its shape,

ix) Presence of any artificial or natural storage such as dams, lakes etc. in the catchment.

x) Possibility of any change in the nature of the catchment due to aforestation, deforestation etc.

(b) *Geological Data* for the particular selected bridge site shall be as below :

i) The nature and properties of the existing soil in bed, banks and approaches.

ii) The details of the trial pits or bore hole sections showing the levels, nature and properties of the various strata to a sufficient depth below the bed level which is suitable for the bridge foundation.

iii) The safe allowable pressure of the foundation soil.

iv) Liability of the site to seismic disturbance and its magnitude.

2.4 SOIL INVESTIGATION.

2.4.1 In order to determine the nature of the soil through which the substructure and the foundation of the proposed bridge will pass and on which the foundations will rest, the soil investigation is necessary. Both preliminary followed by detailed soil investigation are generally undertaken at each proposed bridge pier and abutment location. Disturbed as well as undisturbed soil samples are to be collected and from these soil samples the nature of the soil and its characteristic properties are to be determined. The particle size of the soil strata of the bed upto certain depth will indicate the Lacey's silt factor upon which the depth of foundation from scour consideration is to be fixed. The nature of soil, viz. clay, stiff clay, sand, sand mixed with shingles/gravels, sand mixed with shingles/boulders etc. will indicate the difficulty which might be encountered during the driving of the piles or sinking of the wells. The characteristic properties of the soil such as cohesion, angle of internal friction, unit weight etc. of the soil strata at the side of the piles/wells will give the capacity of the piles or the passive resistance of the soil to be offerd to the well foundation respectively. Co-efficient of compression of the soil around or below the pile/well foundations will predict the settlement characteristics of the foundation. Therefore, a properly carried out soil investigation gives many important information for the design and construction of bridge foundations.

2.4.2 The necessity of conducting soil investigation is to get the following information :

i) Nature of the soil deposit upto sufficient depth (usually 1.25 to 1.5 times the proposed depth of foundation from bed level).

ii) Thickness and composition of each soil layer.

iii) Thickness and composition of rock if rocky strata is available.

iv) The engineering properties of the soil and the rock strata required for the design of the bridge foundation.

2.4.3 Depth of Soil Exploration

2.4.3.1 On the basis of the hydraulic and scour calculation, tentative depth of foundation is fixed as outlined in Chapter 3 subject to the availability of suitable soil strata at this depth and some depth below. The depth of borings shall be the anticipated depth of foundation plus one and a half to twice the width of the foundation (in case of pile foundation, one and a half to twice the width of the pile group). This depth may be increased further if very weak or compressible strata at greater depth is indicated by the preliminary borings.

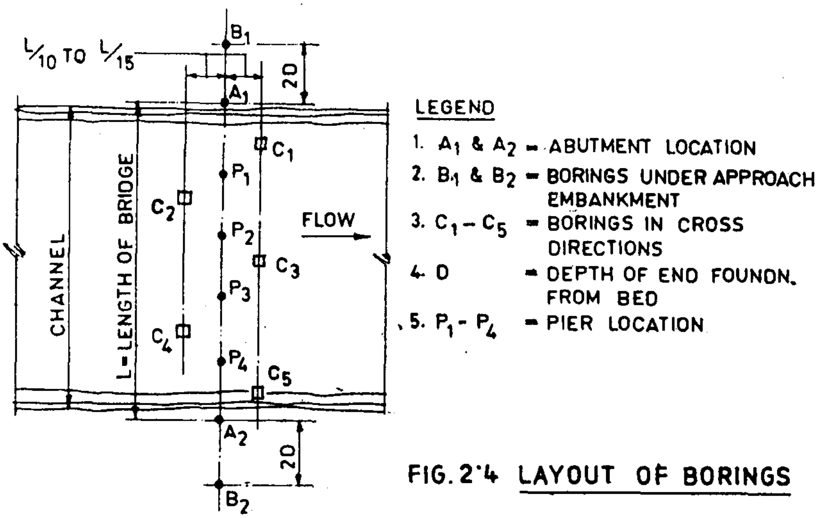


FIG. 2·4 LAYOUT OF BORINGS

2.4.4 Layout of Borings

2.4.4.1 The bore-holes shall be located along the centre line of the proposed bridge at each pier and abutment foundations. In addition, in order to get some information about the change of soil strata on either side of the centre line of the bridge, few more borings are also taken as shown in Fig. 2.4. For the determination of the influence of approach embankment on the end foundation, two more borings B_1 and B_2 on either ends of the bridge at a distance of twice the depth below bed of the last foundation shall be taken.

2.4.5 Bore-log & Sub-soil Profile

2.4.5.1 Bore-logs are prepared on the basis of the soil boring results indicting therein nature of soil, depth and level of each layer starting from bed or ground level. Separate bore log is to be prepared for each bore hole. A typical bore-log is shown in Fig. 2.5. From the results of all the bore-logs, sub-soil profile as shown in Fig. 2.6 shall be drawn. This sub-soil profile will give a comprehensive idea of the nature of soil layers around and below the foundations of the entire bridge.

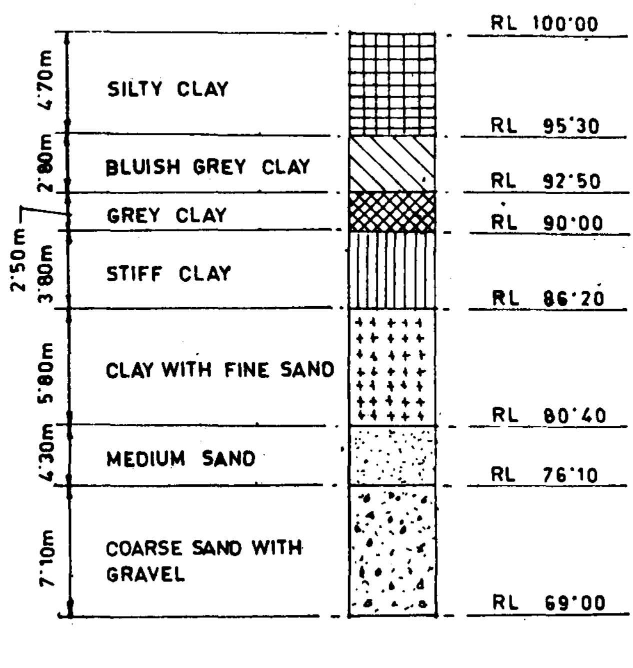


FIG. 2·5 TYPICAL BORING CHART

2.4.6 Borings for Soil Exploration

2.4.6.1 Borings shall be done by any of the following methods :

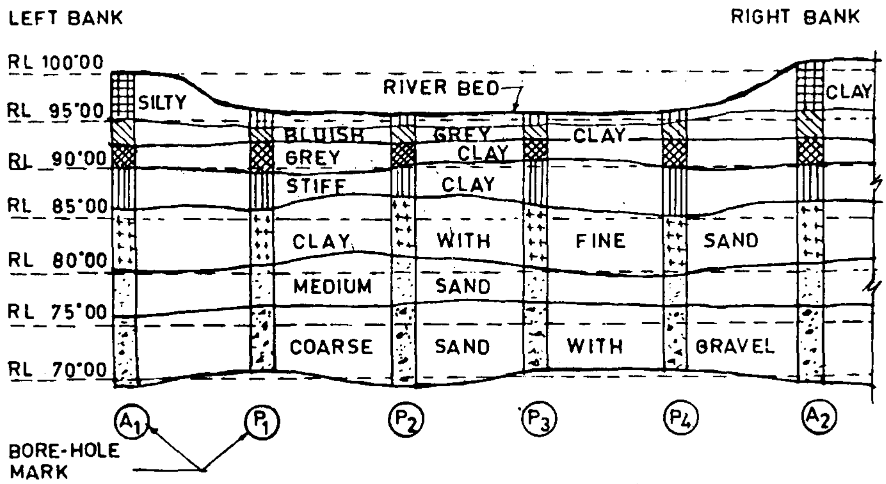


FIG. 2'6 SUB - SOIL PROFILE

i) Auger boring ; ii) Wash boring ; iii) Shell & Auger boring ; iv) Percussion boring ; v) Rotary boring.

2.4.7 Method of Sampling

2.4.7.1 Two types of soil samples shall be taken viz. (i) Disturbed samples and (ii) Undisturbed samples. The method of sampling are given in Table 2.1.

TABLE 2.1 METHOD OF SAMPLING (IRC)[2]

Nature of material	Type of sample		
Soil	Disturbed	i)	Hand samples
		ii)	Auger samples
		iii)	Shell samples
	Undisturbed	i)	Hand samples
		ii)	Tube samples
Rock	Disturbed	Wash samples from percussion rotary drill	
	Undisturbed	Cores	

2.4.8 Tests for Soil Samples

2.4.8.1 *Cohensionless Soil*

a) Classification Tests, density etc. ;

b) Field Test

i) Plate Load Test ; ii) Dynamic Penetration Test

c) Laboratory Tests

i) Shearing strength — Triaxial.

2.4.8.2 *Cohesive Soil*

a) Classification Test, density etc.

b) Field Test

i) Plate Load Test ; ii) Unconfined Compression ;

iii) Vane Shear Test ; iv) Static Cone Penetration Test

c) Laboratory Test

i) Shearing Strength — Triaxial ; ii) Consolidation Test.

2.5 SURVEY OF LOCALLY AVAILABLE MATERIALS

2.5.1 A survey may be conducted in the locality of the proposed bridge site regarding the availability of materials. This information may help in deciding the type of bridge to be adopted in the particular case from economic consideration. Generally, the contractor for the job is responsible for the procurement of the raw materials such as bricks, stone chips, sand etc., but it is a good planning if the sources of materials are ascertained beforehand at the time of preliminary design. This may help in using the proper materials available in the locality at cheaper rate and thus reducing the cost of the project. For example, if in a bridge project well foundation is proposed and if good quality bricks are available at comparatively cheap rate, this material shall have the first preference. On the otherhand, if stone materials are cheaper and there is no difficulty in procuring cement, the use of mass concrete in well steining in such cases is the proper solution. It is, therefore, necessary to mention in the tender documents at the time of inviting tender, the sources of all the building materials so that it may be possible for the tenderer to correctly assess the cost of the materials and quote his rate accordingly. This information will help a good deal in effecting considerable economy.

2.6 REFERENCES

1. IRC : 5-1985, Standard Specifications and code of Practice for Road Bridges, Section I — General Features of Design (Sixth Revision) — Indian Roads Congress.
2. IRC : 78-1983, Standard Specificaitons and Code of Practice for Road Bridges, Section VII — Foundations and Substructure (First Revision) — Indian Roads Congress.
3. Pocket Book for Bridge Engineers — Indian Roads Congress.
4. Murty, V. N. S. — *"Soil Mechanics & Foundation Engineering (Second Edition)"* — Dhanpat Rai & Sons, Delhi - 6.
5. Wintercorn, H. F. and Hsai-Yang Fang — *"Foundation Engineering Hand Book"* — Galgotia Book Source, New Delhi - 1.

Chapter 3

ESTIMATION OF DESIGN DISCHARGE, SCOUR DEPTH ETC AND FIXATION OF WATERWAY, TYPE AND DEPTH OF FOUNDATIONS

3.1 GENERAL

3.1.1 The length of a bridge, depth of foundation, river training works etc. are dependent on the maximum recorded quantum of water or flood discharge which has passed through the river or the channel over which the bridge is proposed and as such the design discharge is very important not only from economic consideration but also from safety or stability consideration. Therefore, the design discharge, which might be the recorded discharge during the past 50-100 years, shall be ascertained very carefully. For this purpose, both the theoretical and practical approach shall be attempted to estimate the maximum flood discharge, these values are compared and then the design discharge is judiciously fixed.

3.1.2 There are various methods for the estimation of flood discharge as outlined below :

i) Catchment-Run-off Method from rainfall and other characteristics of the catchment by the use of empirical formulae or by Rational Method.

ii) From hydraulic characteristics of the stream such as cross-sectional area and slope of the stream.

iii) From area of cross-section and velocity as observed on the stream at the bridge site.

iv) From recorded flood discharge near the bridge site.

3.2 CATCHMENT-RUN-OFF METHOD FOR THE ESTIMATION OF FLOOD DISCHARGE

3.2.1 The catchment area is the command area of a river wherefrom the river gets the supply of water. The catchment area is computed from the contour map and the flood discharge is estimated from the ***"Run-off"*** formula.

3.2.2 The rainfall is measured by rain gauges in millimetre. From the daily record of rainfall, annual rainfall for a zone is determined. The annual rainfall varies from place to place and therefore, the recorded rainfall for a considerable period, say fifty years, is very useful in getting the maximum rainfall recorded during this period.

The estimation of maximum flood discharge shall be based on this maximum recorded rainfall. Table 3.1 gives the rainfall record in different parts of the Indian Union for a period of 15 years (1935-1949).

TABLE 3.1 ANNUAL RAINFALL IN MILLIMETRES IN DIFFERENT PARTS OF INDIA DURING THE PERIOD FROM 1935 TO 1949 *(Khadikar)*[5]

Location	Minimum Annual Rainfall			Maximum Annual Rainfall			Average Annual Rainfall		
	Period 1935-39	Period 1940-44	Period 1945-49	Period 1935-39	Period 1940-44	Period 1945-49	Period 1935-39	Period 1940-44	Period 1945-49
Andhra Pradesh	613	449	755	1179	993	1026	838	777	884
Assam	2283	2465	2627	2659	2757	2993	2490	2586	2771
Bombay Deccan	656	673	702	962	931	1077	780	815	897
Central India (East)	1082	670	999	1243	1224	1436	1108	960	1177
Central India (West)	794	688	910	969	1223	1295	896	1013	1132
Gujarat	538	796	437	1041	1183	1241	729	954	902
Karnataka	799	864	729	969	1125	1181	887	991	931
Orissa	1249	1443	1300	1911	1712	1642	1499	1564	1447
Punjab	169	211	138	640	873	698	378	455	613
Rajasthan	162	340	255	930	1014	907	300	683	252
Tamilnadu	582	459	498	1035	1210	1250	898	844	889
Uttar Pradesh	799	589	933	1459	1110	1276	1016	915	1050
West Bengal	1535	1674	1720	2101	2230	2106	1905	1865	1863

3.2.3 Run-off is defined as the proportion of water out of the total rainfall in the catchment area running to the water course, channel or river. It is needless to mention that the full quantity of rainfall does not reach the water course as some quantity is soaked in the soil to form the sub- soil water strata, some quantity is absorbed by vegetation, some quantity is evoporated and the rest only flows to the channel or river. How the rain water reaches the channel or the river from the catchment area is shown in Fig. 3.1 and Fig. 3.2.

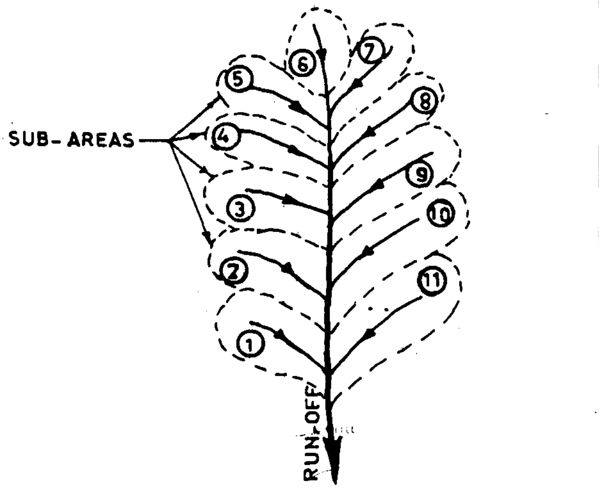


FIG. 3·1 RUN-OFF FROM NORMAL SINGLE CATCHMENT (Deo)[5]

3.2.4 The catchment area of the stream or river upstream of the bridge site is obtained by marking the ridge line of the contour map and measuring the area enclosed by this ridge line with the help of a planimeter or tracing paper graphs.

3.2.5 The possibility of intensive rainfall falling simultaneously over the entire area of a big catchment is less and therefore, a lesser percentage of run-off may be taken. Another important factor which determines the percentage of run- off is the shape of the catchment. Fig. 3.1 and Fig. 3.2 show two types of catchment. In normal single catchment, the watershed is long and narrow having a number of short tributaries joining the main stream. In such catchment, storms of shorter duration which cause the maximum flood discharge, will not reach the bridge site nearly at the same time and as such run-off in such catchment area will be less than that in a fan-like shape of catchment. In the latter case, the tributaries are longer and few in number and therefore, their run-off will reach the bridge site almost simultaneously causing thereby concentration of flow during storms of shorter duration. Hence, even if the catchment area, quantity, duration of rainfall etc. are the same for both types of catchment, the run-off at the bridge site will be more for fan shaped catchment than for normal single catchment.

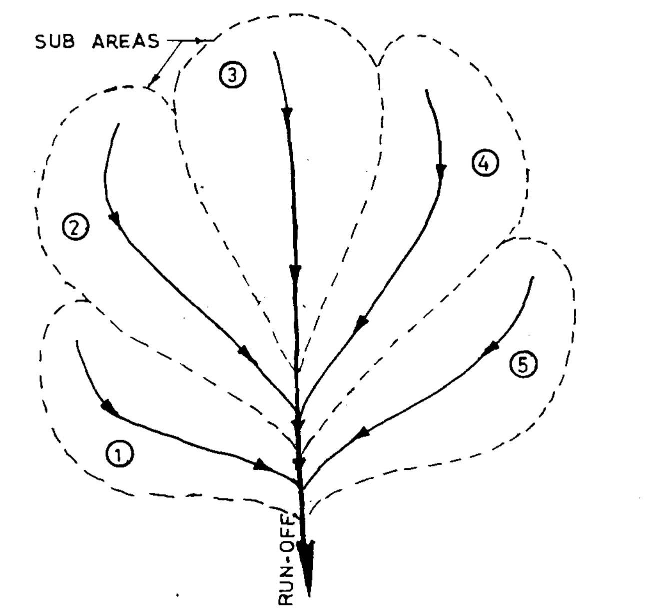


FIG. 3·2 RUN-OFF FROM FAN-LIKE SHAPE OF CATCHMENT (Deo)[5]

3.2.6 Percentage run-off varies from 20 percent to 70 percent depending upon the shape and nature of the catchment. Porosity of soil ; that is, whether sandy, clayey or rocky; degree of previous saturation; area covered by forest; presence of lakes, ponds, swamps, artificial reservoir etc.; determine the percentage run-off. Therefore, while estimating the flood discharge from the catchment area, the aforesaid factors shall be duly taken into consideration.

3.2.7 As discussed before, the run-off depends on the following factors :

i) Degree of porosity and degree of saturation of the soil in the catchment area.

ii) The shape and slope of the catchment area.

iii) Obstacles to flow such as roots of trees, bushes etc.

iv) Degree of vegetation.

v) State of cultivation.

vi) Amount of evaporation.

vii) Intensity of rainfall; Run-off is more if the same amount of rainfall say 50 mm is within a very short period of, say, two hours than is spread for a larger period of, say, 24 hours in which case it is in the form of drizzling.

viii) Total quantity of rainfall in the catchment area.

3.3 ESTIMATION OF FLOOD DISCHARGE BY THE USE OF EMPIRICAL FORMULAE

3.3.1 The flood discharge can be evaluated by using various empirical formulae involving area of the catchment and some coefficient depending upon the location of the catchment.

i) ***Dickens' Formula***

This formula (originally devised for Northern India but can now be used in most of the states of India with the modification of the value of the coefficient C) is given by :

$$Q = C(A)^{\frac{3}{4}} \qquad (3.1)$$

Where Q = Maximum flood discharge in m^3/sec.

A = Area of catchment in sq.km.

And C = A coefficient having value of 11 for Northern India, 14 to 19 for Central India and 22 for Western India.

ii) ***Ryve's Formula***

This formula originally devised for Madras (Tamilnadu) State is given by :

$$Q = C(A)^{\frac{2}{3}} \qquad (3.2)$$

Where, Q and A are the same as in Dickens' formula.

And C = A coefficient equal to 6.8 for areas within 25 km. of the coast, 8.5 for areas between 25 to 160 km. of the coast and 10.0 for limited areas near the hills.

iii) ***Inglis' Formulae***

These formulae used in the state of Maharashtra are given by :

(a) For small areas only

$$Q = 125\sqrt{A} \qquad (3.3)$$

(b) For area between 160 to 1000 sq.km.

$$Q = 125\sqrt{A} - 2.60\,(A - 260) \qquad (3.4)$$

(c) For all types of catchment

$$Q = \frac{125A}{\sqrt{A + 10}} \qquad (3.5)$$

ILLUSTRATIVE EXAMPLE 3.1

The area of a catchment is 800 sq.km. The area is located in Western India within 150 km. from coast. Estimate the maximum flood discharge by using the various empirical formulae and compare the flood discharges.

Solution

(a) Using Dickens' Formula (Equation 3.1)

$$Q = C(A)^{\frac{3}{4}}$$

Where A = 800 sq.km. and C = 22

$$\therefore Q = 22\,(800)^{\frac{3}{4}} = 22 \times 150.4 = 3310 \text{ cum/sec.}$$

(b) Using Ryve's Formula (Equation 3.2)

$$Q = C(A)^{\frac{2}{3}} = 8.5\,(800)^{\frac{2}{3}} = 8.5 \times 86.18 = 733 \text{ cum/sec.}$$

This formula is applicable for Madras (Tamilnadu) State only and as such gives low value which is not considered.

(c) Inglis' Formulae (Equation 3.4 & 3.5)

Using equation 3.4

$$Q = 125\sqrt{A} - 2.6(A-260) = 125 \times \sqrt{800} - 2.6(800-260) = 3536 - 1404 = 2132 \text{ cum/sec.}$$

Using equation 3.5

$$Q = \frac{125A}{\sqrt{A+10}} = \frac{125 \times 800}{\sqrt{800+10}} = \frac{100{,}000}{28.46} = 3514 \text{ cum/sec.}$$

Comparison of flood discharges worked out by various empiral formulae.

	Formula	Max. Flood discharge in cum/sec.
a)	Dickens'	3310
b)	Inglis' (By equation 3.4)	2132
c)	Inglis' (By equation 3.5)	3514

3.4 ESTIMATION OF FLOOD DISCHARGE BY RATIONAL METHOD

3.4.1 If R is the total rainfall in cm for a duration of T hours then the mean intensity of rainfall, I in cm per hour taken over the total duration of the sotrm is given by

$$I = \frac{R}{T} \tag{3.6}$$

3.4.2 For a small time inverval, t, the intensity of rainfall, i, may be more as may be evident from Fig. 3.3 since the mean intensity for a small time interval, t, is more than the mean intensity for the whole time period, T.

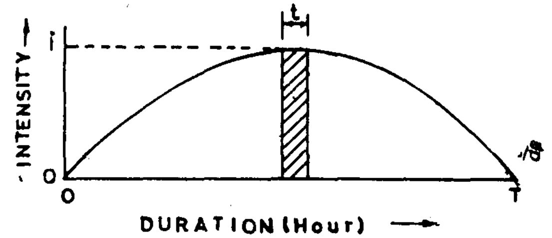


FIG. 3·3 DURATION – INTENSITY OF RAINFALL

3.4.3 The relation between i and I may be shown as :

$$\frac{i}{I} = \frac{T+C}{t+C} \tag{3.7}$$

Where C is a constant and may be taken as unity for all practical purpose.

$$\therefore i = I\left(\frac{T+1}{t+1}\right) \tag{3.8}$$

If t = one hour and corresponding i is taken as i_o and the value of I is taken from equation 3.6

$$\text{then } i_o = I\left(\frac{T+1}{2}\right) = \frac{R}{T}\left(\frac{T+1}{2}\right). \tag{3.9}$$

3.4.4 From equation 3.9, i_o (One hour rainfall) can be worked out if the total rainfall R and duration of the severest storm are known. It is advisable to consider a number of heavy storms spread over a prolonged period and i_o may be calculated for each case and the maximum value of i_o shall be taken as the one hour rainfall of the region for the estimation of flood discharge. From a record of the Meteorological Department, Govt. of India, the values of i_o for various places of the Indian Union are reproduced in Table 3.2

TABLE 3.2 ONE HOUR RAINFALL, i_o (mm) FOR VARIOUS PLACES *(Khanna)*[6]

Sl. No.	Place	i_o (mm)	Sl. No.	Place	i_o (mm)
1	Agartala	66	2	Ahmedabad	80
3	Aligarh	51	4	Allahabad	65
5	Amin Devi	53	6	Amritsar	74
7	Anantpur	38	8	Asansol	86
9	Aurangabad	61	10	Bagdogra	70
11	Bangalore (Aerodrome)	58	12	Bangalore (Central)	61
13	Barakachar	51	14	Barahkshetra	89
15	Barhi	56	16	Baroda	71
17	Barrackpore	58	18	Bhimkund	60
19	Bhopal	72	20	Bhubaneswar	46
21	Bhuj	49	22	Bishungarh	60
23	Bokaro	58	24	Bombay (Kolaba)	129
25	Bombay (Santa Cruz)	91	26	Calcutta (Alipore)	62
27	Chambal	84	28	Cherapunji	127
29	Coimbatore	80	30	Dhanbad	74
31	Dholpur	47	32	Dibrugarh (Mohanbari)	93
33	Dum Dum	68	34	Durgapur	90
35	Gangtak	81	36	Gauhati	61
37	Gaya	70	38	Gwalior	63
39	Hazaribag	78	40	Hirakud	82
41	Hyderabad	102	42	Imphal	48
43	Indore	60	44	Jabalpur	77
45	Jagdalpur	73	46	Jaipur	55
47	Jamshedpur	62	48	Jawai Dam	98
49	Jharsuguda	77	50	Jadhpur	60
51	Tonk Dam	46	52	Kathmandu	44

TABLE 3.2 ONE HOUR RAINFALL, I_o (mm) FOR VARIOUS PLACES *(Khanna)*[6] [Contd.]

Sl. No.	Place	I_o (mm)	Sl. No.	Place	I_o (mm)
53	Kodaikanal	83	54	Konar	59
55	Lucknow	70	56	Madras (Meenambakkam)	62
57	Madras (Nungambakkam)	75	58	Mahabaleswar	51
59	Maithon	54	60	Mangalore	72
61	Marmagao	60	62	Minicoy	70
63	Nagpur	78	64	New Delhi	79
65	North Lakhimpur	65	66	Okha	76
67	Okhaldanga	52	68	Palgauj	56
69	Panaji	54	70	Panambur	37
71	Panchat Hill	71	72	Pathankot	68
73	Patna	59	74	Pokhara	62
75	Pune	47	76	Port Blair	61
77	Punasa	75	78	Raipur	49
79	Ramgarh	56	80	Sagar Island	94
81	Shillong	43	82	Sindri	50
83	Sonepur	78	84	Srinagar	22
85	Shanti Niketan	49	86	Tezpur	63
87	Tiliava Dam	80	88	Tiruchirapalli	78
89	Trivandrum	96	90	Visakhapatnam	63

3.4.5 ***Time of concentration*** is defined as the time taken by the run-off to reach the bridge site from the furthest point of the catchment which is termed as the critical point. Since the time of concentration is dependent upon the length, slope and the roughness of the catchment, a relationship is established with these factors as below :

$$T_c = \left[\frac{0.89\,L^3}{H}\right]^{0.385} \tag{3.10}$$

Where T_c = Concentration time in hours.
H = Fall in level from the critical point to the site of the bridge in metres.
L = Distance from the critical point to the site of the bridge in km.

The values of H and L can be found from the contour map of the catchment area.

The critical intensity of rainfall, I_c, corresponding to the concentration time, T_c, is derived from equation 3.9 considering $I = I_c$ corresponding to $T = T_c$.

$$i_o = I\left(\frac{T+1}{2}\right) = I_c\left(\frac{T_c+1}{2}\right)$$

$$\therefore I_c = i_o \left(\frac{2}{T_c + 1} \right) \tag{3.11}$$

3.4.6 *Estimation of Run-off.* One centemetre of rainfall over an area of one hectare gives a run- off of 100 cu.m per hour. Therefore, a rainfall of I_c cm per hour over an area of A hectare will cause a run-off of 100 A I_c cu.m per hour. If losses due to absorption etc. is considered then the run-off is given by :

$$Q = 100\ PI_cA \text{ cu.m per hour}$$
$$= 0.028\ PI_cA \text{ cu.m/sec} \tag{3.12}$$

Where P = Coefficient depending on the porosity of soil, vegetation cover, initial state of saturation of soil etc. The values of P for various conditions of the catchment area are given in Table 3.3.

TABLE 3.3 VALUES OF P IN EQUATION 3.12 *(IRC)*[1]

Sl.No.	Characteristics of the catchment	Value of P
1	Steep bare rock and city pavements	0.90
2	Steep but wooded rock	0.80
3	Plateau lightly covered	0.70
4	Clayey soils, stiff and bare	0.60
5	Clayey soils lightly covered	0.50
6	Loam lightly cultivated and covered	0.40
7	Loam largely cultivated	0.30
8	Sandy soil, light growth	0.20
9	Sandy soil, heavy brush	0.10

3.4.7 In addition to the coefficient, P, another coefficient, f, is introduced in the formula for calculating the run-off. As the catchment area gets larger and larger, the possibility of reaching the run-off to the bridge site simultaneously from all parts of the catchment is less and less and as such the value of f is gradually reduced as the catchment area is increased. Table 3.4 gives the value of f in equation 3.13 derived from equation 3.12 with the introduction of the coefficient, f, therein.

$$Q = 0.028\ PfI_cA \text{ cu.m/sec.} \tag{3.13}$$

TABLE 3.4 VALUES OF f IN EQUATION 3.13 *(IRC)*[1]

Sl. No.	Area of catchment in hectares	Value of f
1	Nil	1.0
2	4,000	0.85
3	8,000	0.76
4	12,000	0.70
5	16,000	0.67

TABLE 3.4 VALUES OF f IN EQUATION 3.13 (IRC)[1] [Contd.]

Sl. No.	Area of catchment in hectares	Value of f
6	20,000	0.65
7	40,000	0.62
8	80,000 and above	0.60
N.B. The values of f for intermediate areas may be interpolated		

ILLUSTRATIVE EXAMPLE 3.2

The catchment area of a river is 800 Sq.Km. and is composed of sandy soil with thick vegetation cover. The length of the catchment is 30 Km. and the reduced levels of the critical point and the bridge site are 200 m and 50 m respectively. Find out the peak storm discharge by the Rational Method assuming that the rainfall in 5 hours is 20 cm. What will be the peak discharge if the catchment area is of clayey soil lightly covered or of steep but wooded rock?

Solution

i_o (One hour rainfall intensity) from equation 3.9 $= \frac{R}{T}\left(\frac{T+1}{2}\right)$

In the present case, R = 20 cm. ; T = 5 hours

$$\therefore\ i_o = \frac{20}{5}\left(\frac{5+1}{2}\right) = 12 \text{ cm/hour}$$

From equation 3.10, time of concentration, $T_c = \left[\frac{0.89\,L^3}{H}\right]^{0.385}$

Given L = 30 km. ; H = 200 − 50 = 150 m

$$\therefore\ T_c = \left[\frac{0.89\,(30)^3}{150}\right]^{0.385} = (160.2)^{0.385} = 7.06 \text{ hours}$$

The critical intensity of rainfall, I_c, corresponding to the concentration time, 7.06 hours, is obtained by equation 3.11

$$I_c = i_o\left[\frac{2}{T_c+1}\right] = 12\left[\frac{2}{7.06+1}\right] = 2.98 \text{ cm/hour}$$

Maximum peak run-off, from equation 3.13

$$Q = 0.028\,PfI_cA \text{ cu.m/sec}$$

In the present case for catchment area composed of sandy soil with thick vegetation,

A = 800 sq.km = 80,000 hectares ; P from table 3.3 = 0.10 ; f from table 3.4 = 0.60 ; I_c = 2.98 cm/hour

$$\therefore\ Q = 0.028\,PfI_cA = 0.028 \times 0.10 \times 0.60 \times 2.98 \times 80{,}000 = 400 \text{ cum/sec.}$$

When the catchment area is of clayey soil lightly covered, P from table 3.3 = 0.50, values of A, f and I_c remaining as before.

$$\therefore\ Q = 0.028\,PfI_cA = 0.028 \times 0.50 \times 0.60 \times 2.98 \times 80{,}000 = 2003 \text{ cum/sec.}$$

In case of catchment area with steep but wooded rock, P from table 3.3 = 0.80

$$\therefore Q = 0.028\ PfI_cA = 0.028 \times 0.80 \times 0.60 \times 2.98 \times 80{,}000 = 3204 \text{ cum/sec}$$

3.4.8 Therefore, it may be noted from the illustrative example that the peak run-off is very much dependent on the nature of the catchment, other factors remaining the same and varies from 400 cum/sec to 3204 cum/sec when the degree of porosity and absorption of the catchment area is very high or very low. The Rational Method is, therefore, very realistic and considers all relevant factors which regulate the peak run-off. The empirical formulae as described in Art. 3.3 do not consider these factors except some adjustment in the value of the coefficient C and therefore, are not very much realistic.

3.5 ESTIMATION OF FLOOD DISCHARGE FROM CROSS SECTIONAL AREA AND BED SLOPE

3.5.1 By this method the discharge is calculated from Manning's formula,

$$Q = A.V = A\frac{1.4858}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \tag{3.14}$$

Where A = the area of cross section of the stream measured from H.F.L.
n = the rugosity co-efficient.
R = the hydraulic mean depth and equal to the ratio of cross- sectional area, A, to wetted perimeter, P.
S = the bed slope of the stream measured over a reasonably long distance.

In a stream having non-erodable banks and bed, the shape and the size of the cross-section remain practically the same during a flood as at normal times and therefore, the normal cross-section and the perimeter may be used in calculating the discharge. But in a stream flowing through alluvium region, the cross-sectional area and the perimeter may change during highest floods due to the scouring of the banks and the bed and as such in estimating the maximum flood discharge, the depth of scour has to be ascertained first and the values of the cross-sectional area and the perimeter may then be calculated by taking levels of the bed at certain intervals.

The value of the rugosity co-efficient depends on the nature of the bed and the bank of the stream and proper care is requied to be taken in selecting the right value of this co-efficient in order to get the correct discharge. Some values of the rugosity co-efficient, n, are given in table below for various types of surface conditions.

TABLE 3.5 VALUES OF THE RUGOSITY COEFFICCIENT, n *(IRC)*[1]

Sl.No.	Nature of surface	Condition			
		Perfect	Good	Fair	Bad
1	Clean, straight bank, no rifts or deep pools	0.025	0.0275	0.030	0.033
2	Same as (1) but some weeds and stones	0.030	0.033	0.035	0.040
3	Winding, some pools and shoals, clean	0.035	0.040	0.045	0.050
4	Same as (3) but more ineffective slope and sections	0.040	0.045	0.050	0.055

TABLE 3.5 VALUES OF THE RUGOSITY COEFFICCIENT, n *(IRC)*[1] [Contd.]

Sl.No.	Nature of surface	Condition			
		Perfect	Good	Fair	Bad
5	Same as (3) but some weeds and stones	0.033	0.035	0.040	0.045
6	Same as (4) but stony sections	0.045	0.050	0.055	0.060
7	Sluggish river reaches rather weedy or with very deep pools	0.050	0.060	0.070	0.080
8	Very weedy reaches	0.075	0.100	0.125	0.150

ILLUSTRATIVE EXAMPLE 3.3

A river has the bed levels at the highest flood at certain intervals as shown in Fig. 3.4. The R.L. of the lowest beds at 500 m upstream and 500 down stream are 107.42 m and 105.30 m respectively. Calculate the maximum flood discharge if the river has a fairly clean, straight banks but having some weeds and stones.

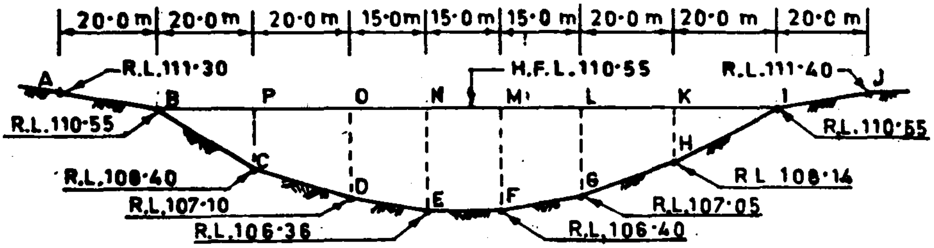


FIG. 3·4 RIVER CROSS-SECTION AT H.F.L. (Example 3·3)

Solution

Area of cross-section A at H.F.L. may be found out by dividing the area into strips such as BPC, PCDO, ODEN etc.

Depth PC =	$110.55 - 108.40$	$= 2.15$ m
Depth OD =	$110.55 - 107.10$	$= 3.45$ m
Depth NE =	$110.55 - 106.36$	$= 4.19$ m
Depth MF =	$110.55 - 106.40$	$= 4.15$ m
Depth LG =	$110.55 - 107.05$	$= 3.50$ m
Depth KH =	$110.55 - 108.14$	$= 2.41$ m
Area BPC =	$\frac{1}{2} \times 2.15 \times 20$	$= 21.50\ m^2$
Area PCDO =	$\frac{1}{2}(2.15 + 3.45) \times 20$	$= 56.00\ m^2$
Area ODEN =	$\frac{1}{2}(3.45 + 4.19) \times 15$	$= 57.30\ m^2$

Area NEFM =	$\frac{1}{2}(4.19 + 4.15) \times 15$	$= 62.55\ m^2$
Area MFGL =	$\frac{1}{2}(4.15 + 3.50) \times 15$	$= 57.37\ m^2$
Area LGHK =	$\frac{1}{2}(3.50 + 2.41) \times 20$	$= 59.10\ m^2$
Area KHI =	$\frac{1}{2} \times 2.41 \times 20$	$= 24.10\ m^2$
	Total Area :	$= 337.92\ m^2$

The wetted perimeter P at H.F.L. is the bed line BCDEFGHI which is the summation of the length of line BC, CD, DE etc. These length may be worked out as below (See Fig. 3.5).

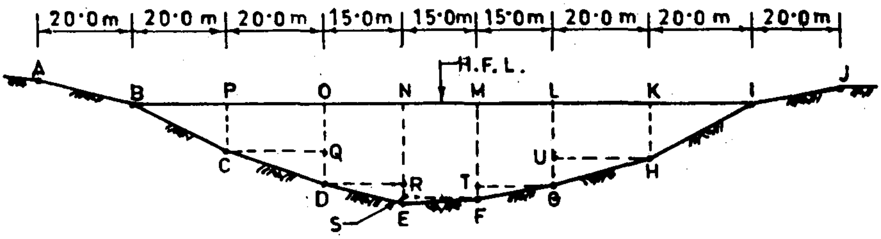


FIG. 3·5 DETERMINATION OF WETTED PERIMETER (Ex. 3·3)

Depth PC = 2.15 m as before

Depth QD = OD – PC = 3.45 – 2.15 = 1.30 m

Depth RE = NE – OD = 4.19 – 3.45 = 0.74 m

Depth SE = NE – MF = 4.19 – 4.15 = 0.04 m

Depth TF = MF – LG = 4.15 – 3.50 = 0.65 m.

Depth UG = LG – KH = 3.50 – 2.41 = 1.09 m

Depth KH = 2.41 m as before.

Length BC $= \sqrt{BP^2 + PC^2} = \sqrt{(20.0)^2 + (2.15)^2} = 20.12$ m

Length CD $= \sqrt{CQ^2 + QD^2} = \sqrt{(20.0)^2 + (1.30)^2} = 20.04$ m

Similarly, DE $= \sqrt{(15.0)^2 + (0.74)^2} = 15.10$ m ; EF $= \sqrt{(15.0)^2 + (0.04)^2} = 15.00$ m

FG $= \sqrt{(15.0)^2 + (0.65)^2} = 15.02$ m ; GH $= \sqrt{(20.0)^2 + (1.09)^2} = 20.03$ m

HI $= \sqrt{(20.0)^2 + (2.41)^2} = 20.14$ m

Hence wetted perimeter, P = 125.45 m

$$\therefore \text{ Hydraulic mean depth, } R = \frac{A}{P} = \frac{337.92}{125.45} = 2.69 \text{ m}$$

Bed slope, S, is the level difference of the lowest bed at 500 m upstream and 500 m downstream divided by the distance

$$\text{i.e., } S = \frac{107.42 - 105.30}{1000} = \frac{2.12}{1000} = 0.0021$$

The rugosity coefficient, n, from Table 3.5 for fairly clean, straight banks but having some weeds and stone is 0.035.

∴ Maximum flood discharge from equation 3.14 is given by

$$Q = A\frac{1.4858\, R^{\frac{2}{3}} S^{\frac{1}{2}}}{n} = 337.92 \times \frac{1.4858 \times (2.69)^{\frac{2}{3}} (0.0021)^{\frac{1}{2}}}{0.035}$$

$$= 337.92 \times 3.76 = 1270 \text{ cum/sec.}$$

3.6 ESTIMATION OF FLOOD DISCHARGE FROM AREA OF CROSS-SECTION AND VELOCITY AS OBSERVED AT BRIDGE SITE

3.6.1 The area of cross-section is measured (as explained in Art. 3.5) by taking a series of levels of the river at H.F.L. at certain intervals. The velocity in this case is determined at site by direct measurement of the velocity in place of theoretical calculation from bed slope etc. as explained in Art. 3.5.

3.6.2 To measure the velocity directly, the river is divided into few sections widthwise and then the velocity for each section is determined by surface float placed at the centre of each section. The time taken by the float to cover a fixed distance is noted by a stop watch and the distance travelled by the float divided by the time taken is the surface velocity of the stream. Such surface velocity is to be determined for each section and weightage average value is obtained for the purpose of flood discharge estimation.

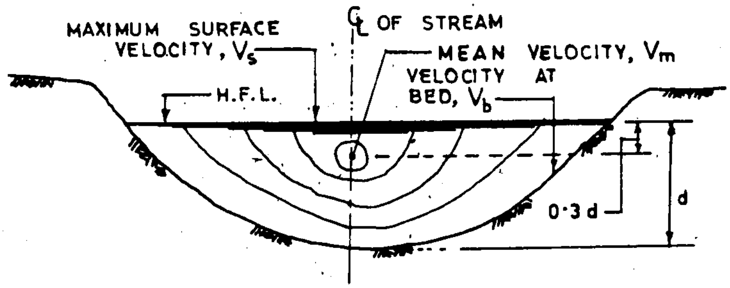


FIG. 3·6 CROSS-SECTION OF A STREAM SHOWING VELOCITY CONTOURS (Deo)[5]

3.6.3 The velocity is least in the vicinity of the bed and banks and mean at the centre line of the stream at a point 0.3 d below the surface where, d, is the depth of water (see Fig. 3.6). If V_s is the velocity at surface, V_b is the velocity at bottom and V_m is the mean velocity then their relationship may be established in the following equation,

$$V_m = 0.7\, V_s = 1.3\, V_b \qquad (3.15)$$

After the determination of the mean velocity of the stream, the flood discharge is obtained by

$$Q = AV_m \qquad (3.16)$$

The velocity of a stream can also be measured by a current-meter which is a very compact instrument.

3.7 FLOOD DISCHARGE FROM AVAILABLE RECORDS

3.7.1 In some cases it may be possible to have the maximum flood discharge measured at weir or barrage sites. This value may be compared with the theoretical worked out value and a final value may be selected. The flood discharge thus obtained, though very realistic, suffers from one drawback viz. the age of the record, since the weirs or the barrages are mostly of recent construction. The flood discharge shall preferably be the maximum of 100 years' recorded value for important bridges and 50 years' recorded value for less important bridges. The terms ***"100 years' value"*** and ***"50 years' value"*** are defined as momentary peak discharge which occur ***"on the average"*** once in 100 years or once in 50 years. The phrase ***"on the average"*** means all the peak discharges as observed over a period of 100 years or 50 years as the case may be and average of the peaks is taken.

3.8 FIXING DESIGN DISCHARGE

3.8.1 The design flood discharges may be ascertained by comparison of the values obtained by various methods as explained before and taking the highest value provided it does not exceed the next higher value by more than 50 percent. The design flood discharge may be limited to 50 percent in excess of the record higher value if the highest value exceeds 50 percent of the record higher value.

The design flood discharge shall be based on 100 years' record for important bridges and 50 years' record for less important bridges as stated before.

3.8.2 For the purpose of keeping adequate margin of safety, the foundation and protection works shall be designed for a larger discharge over the design discharge as obtained before. The percentage increase shall be as given in Table 3.6, the exact value to be fixed by the design engineer.

TABLE 3.6 PERCENTAGE INCREASE OF DESIGN DISCHARGE *(IRC)*[3]

Sl. No.	Catchment area (Sq.Km.)	Percentage increase over the design discharge
1	Upto 500	30
2	Above 500 and upto 5,000	25 to 20
3	Above 5,000 and upto 25,000	20 to 10
4	Above 25,000	10

3.9 NORMAL SCOUR DEPTH — LACEY'S THEORY OF REGIME CONDITION

3.9.1 As explained previously, the bed and the bank levels of a stream having non-erodible strata are the same during flood and after its subsidance but these are not so in case of stream flowing through erodible region having alluvium soil strata. These later streams meander a great deal resulting in wide but shallow cross-section.

It has been observed that alluvium streams which are composed of loose granular materials have a natural tendency to scour away or silt up till it acquires a cross-section and a bed slope such that a ***"non-silting and non-scouring"*** velocity is attained. When such condition of the stream is reached, it becomes stable and the acquired shape of the slope and the cross- section is maintained and this state of the stream is known as ***"regime condition"***.

According to Lacey's Theory, when an alluvium stream carrying a discharge, Q, comes to regime, it has a regime slope, S, a regime hydraulic mean depth, R, and a regime wetted perimeter, P, and therefore, it has a regime velocity, V, and a regime cross- sectional area, A.

3.9.2 The following empirical formula may be used for the estimation of scour for channels flowing in non-coherent alluvium. The mean depth of scour, dm, in metres from H.F.L. is given by

$$dm = 1.34\left[\frac{q^2}{f}\right]^{\frac{1}{3}} \tag{3.17}$$

Where q = Discharge in cum/sec per metre width
f = Silt factor.

The value of q shall be the maximum of the following :

i) Total design discharge divided by the effective linear water-way between abutments or guide bunds.

ii) The value shall be fixed taking into consideration any concentration of flow through a portion of the channel as may be evident from the study of the cross-section of the channel. Such consideration need not be made for minor bridges having length less than 60 metres.

iii) Actual observations, if any.

3.9.3 From equation 3.17 it may be observed that the scour depends on the silt factor which again depends on the size and looseness of the grains of the alluvium soil. The value of silt factor, f, is determined from the formula $f = 1.76\sqrt{m}$ where "m" is equal to the mean diameter in millimetre of soil grains of equivalent sphere. A few values of the silt factor are given below for rough guide.

TABLE 3.7 VALUES OF SILT FACTOR, f *(IRC)*[3]

Bed material		Grain size in mm	Silt factor, f
Silt	Fine	0.081	0.500
	Fine	0.120	0.600
	Fine	0.158	0.700
	Medium	0.233	0.850
	Standard	0.323	1.000
Sand	Medium	0.505	1.250
	Course	0.725	1.500
	Mixed with fine bajri	0.988	1.750
	Heavy	1.290	2.000

3.10 LINEAR WATERWAY FOR BRIDGES

(a) Non-erodible Streams

The linear waterway required for a bridge across a stream having non-erodible banks and bed is the distance between banks at the water surface elevation at H.F.L. Some reduction in the waterway may be possible for some streams with moderate velocities.

(b) Alluvium Stream

The linear waterway across an alluvium stream should normally be kept to the regime width for stability of the bridge structure. The regime width, W, in metres, is given by

$$W = C\sqrt{Q} \tag{3.18}$$

Where Q = Design discharge in cum/sec.
C = Constant usually taken as 4.8 for regime channels but it may vary between 4.5 to 6.3 depending on local conditions.

If the waterway is kept somewhat in excess, the normal scour depth will not be less than the regime depth and therefore, the cost of the foundations will remain unchanged though the cost of the superstructure will go up due to the increase in the length of the bridge. On the other hand, if the linear waterway is reduced from the regime width, excessive scour may take place thus requiring deeper foundations though there may be some economy in the cost of superstructure. It, therefore, transpires that while increased linear waterway will undoubtedly add to the cost, the reduction in the waterway may sometimes reduce the cost if the savings in the superstructure cost is more than the additional cost for foundation. This point needs special and careful consideration before contraction of the stream beyond regime width is envisaged. In the confined and restricted channel, as the scour is more than in the regime channels, not only deeper foundations are required but also expensive stone protections in pitching, apron, toe-walls, cut-off walls and guide bunds etc. are necessary thereby increasing the overall cost. Therefore, there is an economical limit beyond which confining the river should not be attempted.

3.11 EFFECTIVE LINEAR WATERWAY

In determining the scour depth, the effective linear waterway between abutments or guide bunds (and not overall waterway) shall be taken into consideration. The effective linear waterway, L, is the total width of waterway at H.F.L. minus the effective width of obstruction due to each pier and its foundation upto the normal scour level. Effective linear waterway is given by

$$L = B - nt \qquad (3.19)$$

Where B = Overall waterway between abutments or guide bunds at H.F.L. in metres.
n = Nos. of piers.
t = Width of obstruction due to each pier and its foundation upto normal scour level in metres.

3.12 MAXIMUM SCOUR DEPTH FOR FOUNDATION AND GUIDE BUND DESIGN

The design of pier and abutment foundation in a straight reach and having individual foundation shall be based on the following depth of scour :

i) Near piers	2.0 dm
ii) Near abutments	1.27 dm when the approach retained and 2.00 dm when scouring all round.

For the design of raft foundation, shallow foundation and floor protection works, the following scour values shall be considered :

i) In a straight each	1.27 dm
ii) At a moderate bend	1.50 dm
iii) At a severe bend	1.75 dm
iv) At a right-angled bend	2.00 dm

3.13 DEPTH OF FOUNDATION

The depth of foundations for substructures shall be determined from the consideration of the safe bearing pressures of the soil after taking into account the effect of scour as outlined in the proceeding paras.

For bridges in inerodible streams, the foundations shall be securely anchored into the bed materials by taking the foundation in it about one metre or so. The anchoring is usually done by mean of anchor bars.

The foundations for bridges to be built across alluvium streams shall be taken down below the maximum scour level at least one- third of the maximum scour depth from the H.F.L. and not less than 2.0 metres for piers and abutments with arches and 1.2 metres for piers and abutments supporting other types of superstructure. In all cases, the foundations shall be taken down to such depths that there may not be any doubt about their stability and proper grip of the foundation is ensured. The maximum scour near pier foundation is 2.00 dm from H.F.L. as explained in Art. 3.12. Therefore, the grip length for deep foundations shall be ⅔ dm and depth of foundation from H.F.L. shall be $2\frac{2}{3}$ dm = $\frac{8}{3}$ dm.

ILLUSTRATIVE EXAMPLE 3.4

Calculate the linear waterway and mean scour depth if the design discharge is 1000 cubic metres per second and the value of silt factor is 0.8. If the linear waterway is restricted to 100 metres with two intermediate pier foundations having average 2.75 metres obstruction to flow for each pier, what will be the mean scour depth? Also determine the minimum depth of foundation in each case.

Solution

From equation 3.18

$$\text{Linear waterway, } W = C\sqrt{Q} = 4.80\sqrt{1000} = 151.80 \text{ m}$$

From equation 3.17

$$dm = 1.34\left[\frac{q^2}{f}\right]^{\frac{1}{3}}$$

$$q = \frac{1000}{151.80} = 6.59 \text{ cum/sec.}$$

$$\therefore dm = 1.34\left[\frac{6.59^2}{0.8}\right]^{\frac{1}{3}} = 1.34 \times 3.79 = 5.07 \text{ m from H.F.L.}$$

$\therefore$ Maximum scour depth near pier foundation from Art. 3.12

$$= 2.0 \times 5.07 \times = 10.14 \text{ m from H.F.L.}$$

Depth of foundation from H.F.L. (Art. 3.13)

$$= \frac{8}{3} dm = \frac{8}{3} \times 5.07 = 13.52 \text{ m}$$

When the linear waterway is restricted to 100 m, the effective linear water from equation 3.19

$$= B - nt = 100.00 - 2 \times 2.75 = 94.5 \text{ m}$$

$$\therefore Q = \frac{1000}{94.5} = 10.58 \text{ cum/sec.}$$

$$\therefore dm = 1.34\left[\frac{10.58^2}{0.8}\right]^{\frac{1}{3}} = 1.34 \times 5.19 = 6.95 \text{ m}$$

Maximum scour depth near pier foundation

$$= 2.0 \times 6.95 = 13.9 \text{ m from H.F.L.}$$

Depth of foundation from H.F.L. $= \frac{8}{3}\,\text{dm} = \frac{8}{3} \times 6.95 = 18.53$ m

It may be interesting to compare the various values under regime waterway and with restricted waterway as shown below :

		Regime Condition	Restricted Condition
1.	Linear waterway	151.80 m	94.5 m
2.	Discharge per metre	6.59 cum/sec.	10.58 cum/sec.
3.	Mean scour depth from H.F.L.	5.07 m	6.95 m
4.	Maximum scour depth from H.F.L.	10.14 m	13.90 m
5.	Depth of foundation from H.F.L.	13.52 m	18.53 m

3.14 AFFLUX

The afflux is the heading up of flood water on the upstream of a bridge and is measured by the difference in levels between the upstream and downstream water surface of the bridge (Fig. 3.7). When the natural waterway of a stream is obstructed by providing less opening, the afflux occurs. Therefore, in keeping free-board above H.F.L. for such type of bridges, the afflux should be given due consideration. The top level and length of guide bunds and flood protection bunds shall also be fixed with due consideration of afflux. The afflux is calculated using Molesworth's formula as given below :

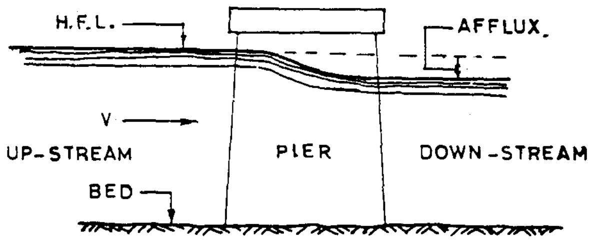


FIG. 3·7 AFFLUX AT A BRIDGE

$$\text{Afflux, H in metres} = \left[\frac{V^2}{17.9} + 0.015\right]\left[\left(\frac{A}{a}\right)^2 - 1\right] \tag{3.20}$$

Where V = Average velocity of river upstream of the bridge prior to restriction in metres/sec.

A = Un-restricted sectional area of the river in sq.m.

a = Restricted sectioinal area of river in sq.m.

ILLUSTRATIVE EXAMPLE 3.5

Calculate the afflux on the upstream of the bridge with the design data as given in the illustrative example 3.4.

Solution

Unrestricted waterway	=	151.8 m
Restricted waterway	=	94.5 m
Mean scour for unrestricted waterway	=	5.07 m
Mean scour for restricted waterway	=	6.95 m

It is assumed that at H.F.L., mean scour for restricted waterway has not reached the ultimate value but an intermediate value i.e., $\frac{1}{2}(5.07 + 6.95) = 6.01$ m

$$A = \text{Area of un-restricted section} = \frac{3}{4} \times 151.8 \times 5.07 = 577.22 \text{ sq.m.}$$

$$a = \text{Area of restricted section} = \frac{3}{4} \times 94.5 \times 6.01 = 425.96 \text{ sq.m.}$$

$$V = \frac{Q}{A} = \frac{1000}{577.22} = 1.73 \text{ m/sec.}$$

$$\therefore \text{Afflux, } H = \left[\frac{V^2}{17.9} + 0.015\right]\left[\left(\frac{A}{a}\right)^2 - 1\right]$$

$$= \left[\frac{1.73^2}{17.9} + 0.015\right]\left[\left(\frac{577.22}{425.96}\right)^2 - 1\right]$$

$$= (0.167 + 0.015)(0.836) = 0.512 \text{ m}$$

3.15 REFERENCES

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CHAPTER **4**

CLASSIFICATION OF HIGHWAY BRIDGES AND SELECTION OF THE TYPE OF BRIDGE FOR ANY PARTICULAR SITE

4.1 CLASSIFICATION OF HIGHWAY BRIDGES

4.1.1 Highway bridges are classified from various considerations such as their life span, purpose, length etc. as indicated below :

A. Life span of the bridge
B. Purpose of the bridge
C. Span of the bridge
D. Load carrying capacity of the bridge
E. Material of construction of the bridge
F. Span arrangement of the bridge
G. Structural arrangement of the bridge

A. Classification according to life span of the bridge

i) Temporary bridge ii) Permanent bridge iii) Semi-permanent bridge

B. Classification according to purpose of the bridge

i) Viaduct ii) Road over bridge iii) Road Underpass iv) Submersible bridge
v) High Level bridge vi) Swing bridge vii) Floating bridge viii) Lift bridge
ix) Flying bridge x) Transporter bridge xi) Traverser bridge xii) Cut-boat bridge
xiii) Bascule bridge

C. Classification according to span of the bridge

i) Minor bridge ii) Major bridge iii) Long span bridge

D. Classification according to load carrying capacity of the bridge

i) Class-3 bridge, Class-5R bridge, Class-9R bridge, Class- 70R bridge etc.

ii) Class B bridge iii) Class A bridge iv) Class AA bridge

E. Classification according to material of construction of the bridge

i) Timber bridge ii) Masonry bridge iii) Reinforced concrete bridge

iv) Prestressed concrete bridge v) Steel bridge vi) Composite bridge

F. Classification according to span arrangement of the bridge

i) Simply supported bridge ii) Continuous bridge

iii) Cantilever bridge iv) Balanced cantilever bridge

G. Classification according to structural arrangement of the bridge

i) Slab bridge ii) Slab and girder bridge iii) Box cell bridge iv) Hollow-box girder bridge

v) Portal frame bridge vi) Arch bridge vii) Plate girder bridge viii) Box girder bridge

ix) Trussed girder bridge x) Cable Stayed bridge xi) Suspension bridge

4.1.2 On the basis of the above classification of bridges, further sub-classification of bridges may be made as stated hereinafter.

4.1.3 Sub-classification of Temporary Bridges

1. *According to the purpose of the bridge*

   A. *Floating bridges*

      i) Pontoon bridge (Fig. 18.17) ii) Boat bridge (Fig. 18.16)

      iii) Raft bridge (Fig. 18.15) iv) Cut-boat bridge (Fig. 18.13)

   B. *Flying bridges*

      i) Using a suspension cable ii) Using anchor and swinging cable

      iii) Using a warp

2. *According to the materials of construction*

   A. *Timber bridges*

      i) Timber trestle support, timber deck over timber beam. (Fig. 18.1)

      ii) Timber trestle support, timber deck over RSJ (Fig. 18.2)

      iii) Timber crib support, timber deck over timber beam.

      iv) Timber crib support, timber deck over RSJ.

4.1.4 Sub-Classification of Permanent Bridges

1. *According to the purpose of the bridge*

   A. *Viaducts*

      i) Slab viaduct ii) Slab and girder viaduct iii) Box girder viaduct

   B. *Road Over bridges*

      i) Slab ROB ii) Slab and girder ROB iii) Box girder ROB

C. *Road Underpasses*

i) Slab underpass ii) Slab and girder underapss

iii) Box girder underpass iv) Portal underpass (Fig. 4.6)

D. *High level bridges*

i) Slab bridge (Fig. 4.1) ii) Slab and girder bridge (Fig. 4.2)

iii) Box girder bridge (Fig. 4.4) iv) Box cell bridge (Fig. 4.5)

v) Arch bridge (Fig. 4.8 to 4.11) vi) Cable stayed bridge (Fig. 17.14 to 17.17)

vii) Suspension bridge (Fig. 17.23 to 17.26)

E. *Swing bridges*

i) Swing bridge with central pivot. (Fig. 18.9a)

ii) Swing bridge with turn table. (Fig. 18.9b)

F. *Lift bridge* (Fig. 18.14)

G. *Transporter bridge* (Fig. 18.12)

H. *Traverser bridge* (Fig. 18.11)

I. *Bascule bridges*

i) Single bascule bridge (Fig. 18.10a)

ii) Double bascule bridge (Fig. 18.10b)

2. ***According to the materials of construction***

A. *Masonry arch bridges*

B. *R. C. bridges*

i) Slab bridge (Fig. 4.1) ii) Slab and girder bridge (Fig. 4.2)

iii) Box cell bridge (Fig. 4.5) iv) Hollow box girder bridge (Fig. 4.4b)

v) Portal frame bridge (Fig. 4.6) vi) Arch bridge (Fig. 4.9 to 4.11)

C. *Prestressed concrete bridges*

i) Slab and girder bridge (Fig. 16.7 & 16.13)

ii) Box girder bridge (Fig. 16.24 & 16.25)

D. *Steel bridges*

i) Arch bridge (Fig. 17.4 & 17.5) ii) Plate girder bridge (Fig. 14.3 & 14.4)

iii) Box girder bridge (Fig. 17.3) iv) Trussed girder bridge (Fig. 17.7 & 17.8)

v) Cable stayed bridge (Fig. 17.14 to 17.17) vi) Suspension bridge (Fig. 17.23 to 17.26)

E. *Composite bridges*

i) R. C. slab over P. S. C. girder (Fig. 4.7b) ii) R. C. slab over steel girder (Fig. 4.7a)

3. ***According to the structural arrangement***

A. *Slab bridges*

i) Simply supported bridge (Fig. 4.1) ii) Continuous bridge (Fig. 4.3a)

iii) Balanced cantilever bridge (Fig. 4.4a)

B. *Slab and girder bridges*

i) Simply supported bridge (Fig. 4.2) ii) Continuous bridge iii) Balanced cantilever bridge

C. *Box Cell bridges*

D. *Hollow-box girder bridges*

i) Simply supported bridge ii) Continuous bridge (Fig. 4.3b)

iii) Balanced cantilever bridge (Fig. 4.4b)

E. *Portal frame bridges*

i) Steel bridge ii) Concrete bridge (Fig. 4.5 & 12.1)

F. *Arch bridges*

(a) Concrete arch bridge

i) Fixed arch bridge (Fig. 4.8 & 4.9) ii) Two hinged arch bridge (Fig. 13.3b)

iii) Three hinged arch bridge (Fig. 13.3c) iv) Tied — arch bridge (Fig. 4.10)

(b) Steel arch bridge with solid rib or trussed rib

i) Fixed arch bridge ii) Two hinged arch bridge

iii) Three hinged arch bridge iv) Tied arch bridge

G. *Plate girder bridges*

i) With concrete deck (Fig. 14.13)

ii) With orthotropic steel plate deck (Fig. 17.3)

H. *Box girder bridges*

i) R. C. box girder bridge (Fig. 4.3b & 4.4b)

ii) Prestressed concrete box girder bridge (Fig. 16.24 & 16.25)

iii) Steel box girder bridge (Fig. 17.2 & 17.3)

I. *Cable-stayed bridges*

i) Concrete deck over plate girder (Fig. 17.17)

ii) Orthotropic steel plate deck over steel box girder. (Fig. 17.14 & 17.15)

iii) Concrete deck over concrete box girder. (Fig. 17.16)

J. *Suspension bridges*

i) Back stay unloaded or loaded

ii) Hangers vertical or inclined (Fig. 17.24 or 17.25)

iii) Decking with steel grid filled with concrete or with orthotropic steel plate. (Fig. 17.24 or 17.23)

4.2 ILLUSTRATIONS

4.2.1 Some types of the bridges as mentioned in the above classification list are illustrated below. Most of the remaining bridges are illustrated in the subsequent chapters while dealing with such bridges. Reference to figure numbers has been indicated in the sub-classification list for such bridges.

4.2.2 Simply supported solid slab bridges

Simply supported solid slab bridges are generally found to be economic for span upto 9.0 metres. These are constructed with reinforced concrete slab of uniform thickness thereby requiring simple shuttering and flasework as well as simple placement of reinforcing bars.

4.2.3 Simply supported slab and girder bridges

Slab and girder bridges (R. C. T-beam) of simply supported span are used for span where solid slab bridges are found uneconomic. Generally 9.0 to 20.0 metre spans may be adopted for this type of bridges.

4.2.4 Continuous span solid slab and slab & girder bridges

Where foundation can be supported on good rock or where foundation soil is such that differential settlement of supports can be eliminated, continuous span superstructure is an ideal solution. In such cases, due to continuity, both the span and support design noments are reduced compared to a simply supported superstructure. Span range for continuous solid slab bridges is between 10.0 to 20.0 metres and for slab & girder bridges is between 20.0 to 40.0 metres Hollow-box continuous structure upto 100 metres span may be possible.

4.2.5 Balanced Cantilever Bridges

Balanced cantilever type superstructure can cover comparatively longer spans than the simply supported superstructures. But unlike continuous spans, slight differential settlements under the pier or abutment foundations are not detrimental to the safety of the structures. Of this type, box-girder, slab and girder or solid slab comes in order of preference so far the capacity for bridging longer spans is concerned. Spans of about 40.0 to 100.0 metres are not uncommon with box-girder bridges while slab and girder bridges of 20.0 to 40.0 metres are usually met with. Solid slab super-structures upto 20.0 metres spans may be used with advantage without any difficulty.

4.2.6 Box-cell Bridges

Solid slab box-cell bridges are used in road underpass or subways. These may also be used in channels where the scour is negligible or in canals where the velocity is non-scouring and non-silting. These type of bridges are used with advantage where the foundation soil near

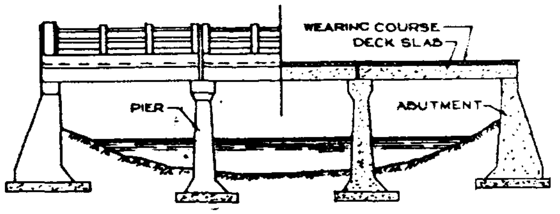


FIG. 4·1 SOLID SLAB BRIDGE

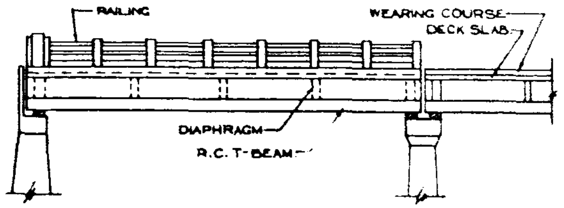
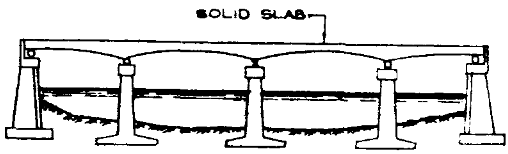
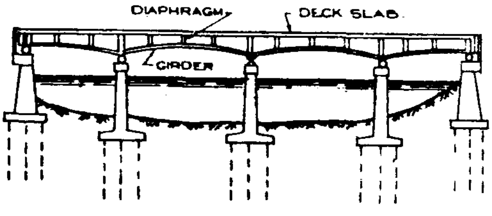


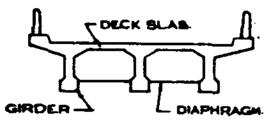
FIG. 4·2 SLAB AND GIRDER BRIDGE



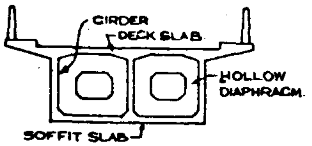
(a) SOLID SLAB TYPE



(b) SLAB AND GIRDER TYPE



(i) CROSS SECTION OF SLAB AND GIRDER BRIDGE.



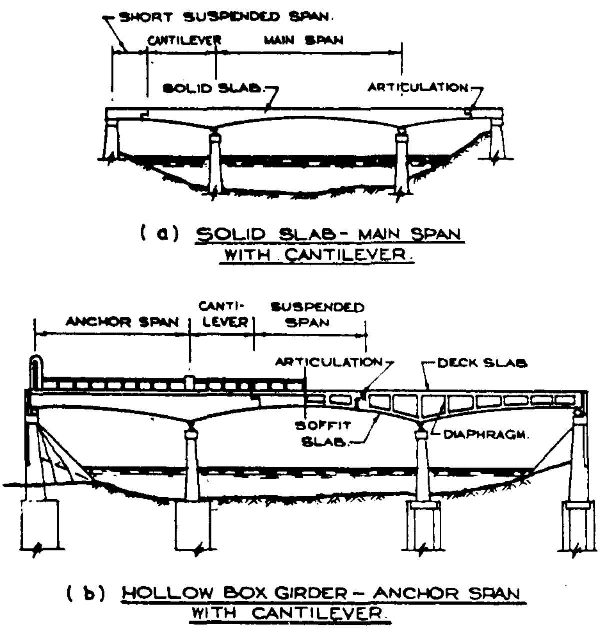
(ii) CROSS SECTION OF HOLLOW BOX GIRDER BRIDGE.

FIG. 4·3 CONTINUOUS BRIDGES

the bed has a poor bearing power since the bottom or base slab cover nearly the entire bridge width and therefore, brings down the actual load per unit area on the soil. The box-cells are either square or nearly square so that the thickness of the deck slab, base slab and the verticals is the same. The span range usually adopted for such type of structures is between 3.0 to 9.0 metres.

4.2.7 Portal-frame bridges

Similar to the continuous span bridges, this type of superstructure need unyielding foundation materials such as good rock on which the foundation can rest otherwise differential settlement may cause harmful effects on the safety of the structures. This is why in ordinary soil, this type of bridge is not suitable. Slab and GIRDER type portal frame superstructure may be found useful for spans between 20.0 to 40.0 metres. The span of solid slab portal frame superstructure should not generally exceed 25.0 metres. These type of structures are ideally suitable for overbridges and underpasses or subways.



(a) SOLID SLAB - MAIN SPAN WITH CANTILEVER.

(b) HOLLOW BOX GIRDER - ANCHOR SPAN WITH CANTILEVER.

FIG. 4·4 BALANCED CANTILEVER BRIDGES

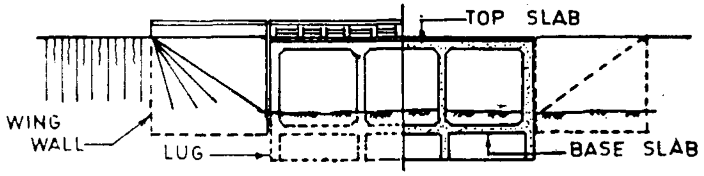


FIG. 4·5 SOLID SLAB BOX-CELL BRIDGE

4.2.8 Composite Bridges

In reinforced concrete slab and girder bridges, the deck slab not only transfers the superimposed load to the supporting girders through transverse bending but also acts as flange of the T-beam to resist longitudinal bending moments. Concrete being good in compression, the deck slab takes nearly the entire compressive force due to longitudinal bending of the girder. In bridges with deck slab simply resting on prefabricated girders, either steel or concrete, no such advantage can be taken unless the cast-in-situ deck slab is made monolithic with the prefabricated girders by some mechanical means.

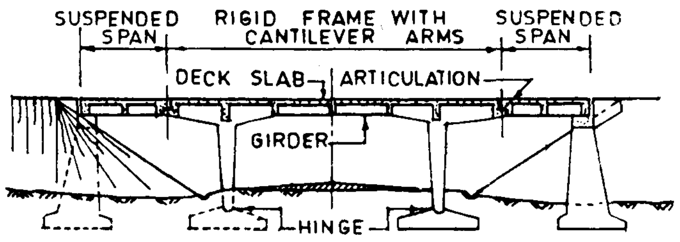


FIG. 4·6 SLAB AND GIRDER RIGID FRAME SUBWAY

This is achieved by the use of ***"Shear connectors"*** which make the two units monolithic.

4.2.9 Arch Bridges

Concrete arch bridges of the following types are constructed :

i) ***Barrel arch — two hinged, three hinged or fixed*** (Fig. 4.8)

ii) ***Open spandrel arch — two hinged, three hinged or fixed.*** (Fig. 4.9)

iii) ***Bow-string girder*** (Fig. 4.10)

iv) ***Ribbed type arch with partially hung decking.*** (Fig. 4.11)

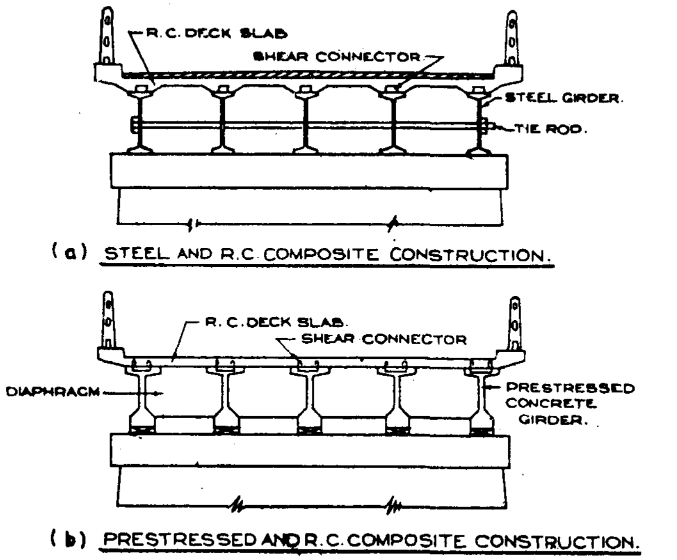


FIG. 4·7 COMPOSITE BRIDGES

4.3 SELECTION OF THE TYPE OF BRIDGE

4.3.1 For any bridge, selection of the type of structure to be adopted requires careful examinations of all the factors governing economy, safety, durability, time of erection, availability of materials and equipments, maintenance cost etc.

As a rule, economy demands that the number of spans should be as small as possible for bridges where difficult conditions are anticipated in the construction of the foundations in addition to incurring extra cost thereby. Moreover, provision of lesser number of piers in the river, improves the flow of water. But it is also to be remembered in deciding the span lengths that longer span means greater cost per unit length of the superstructure. It is, therefore, important to compare the cost of both the superstructure and the substructure including the foundation so that one which is economical and at the same time satisfy other requirements is adopted.

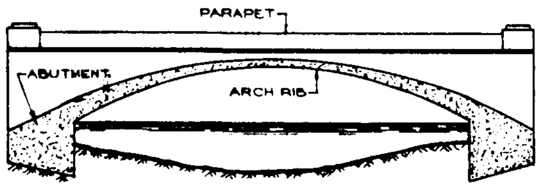


FIG. 4·8 BARREL ARCH BRIDGE (Legat et al)[3]

4.3.2 The selection of the type of bridge to be adopted at a particular site depends on the following consideration :

i) Channel characteristics i.e., bed materials, depth of water during dry season or flood season, tidal variation, scour depth etc.

ii) Hydraulic data viz., velocity, design discharge etc.

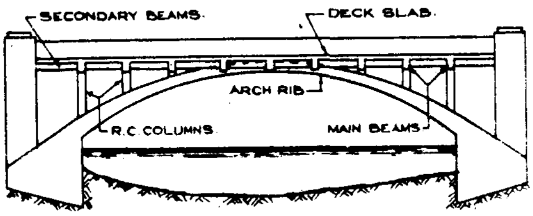


FIG. 4·9 OPEN SPANDREL ARCH BRIDGE (Legat et al)[3]

iii) Subsoil condition and its load bearing capacity.
iv) Frequency and duration of flood.
v) Traffic volume
vi) Navigation requirements
vii) Availability of fund
viii) Availability of labour and materials and their unit cost.
ix) Time period of construction.
x) Transport and erection facilities available
xi) Strategic considerations
xii) Aesthatic considerations
xiii) Maintenance cost.

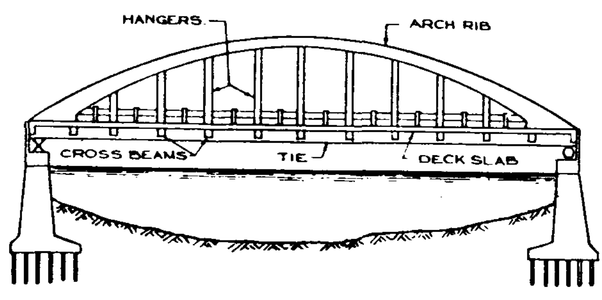


FIG. 4·10 BOWSTRING GIRDER BRIDGE (Legat et al)[3]

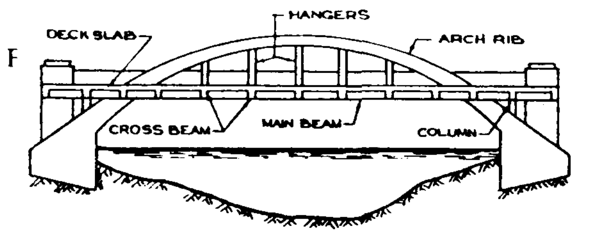


FIG. 4·11 RIBBED ARCH WITH PARTIALLY HUNG DECKING (Legat et al)[3]

4.3.3 Choice between temporary ige and permanent bridge

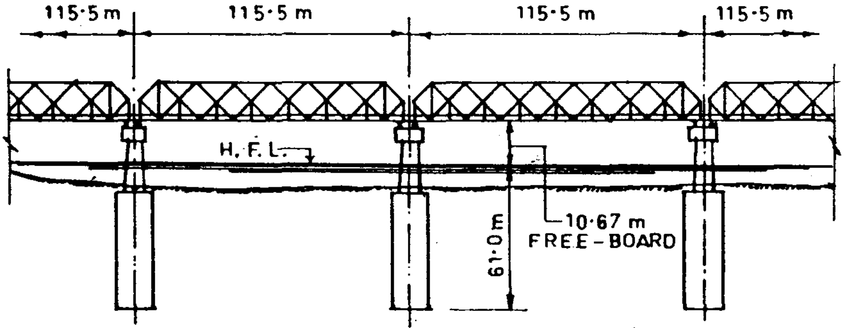
The construction of a permanent bridge requires more fund and therefore, whenever there is shortage of fund, temporary bridge is a short-term solution. Temporary bridge may also be constructed on a less important road where the volume of traffic hardly justifies construction of a permanent bridge at a larger cost. The temporary bridge may be replaced by a permanent bridge when fund position improves or when the increased volume of traffic demands the construction of a permanent bridge.

4.3.4 Choice between submersible bridge and high level bridge

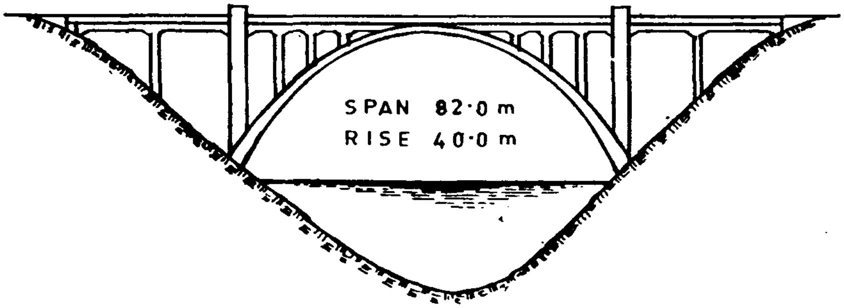
"*Submersible bridges*", as the name implies, remain submerged during high floods and as such traffic has to be kept suspended for few hours or days and for few occasions in a year. Therefore, when sufficient funds are not available, submersible bridges can be constructed over a stream where interruption to traffic movement is as less as possible or where traffic volume is such that such interruption of traffic for few occasions does not affect public interest significantly. It is, therefore, evident that construction of submersible bridge on National Highways or State Highways is not desirable. These bridges may be constructed on village roads or less important district roads. Where temporary bridges or submersible bridges cannot be constructed for public interest considering the volume of traffic, construction of high level bridges is the only choice.

4.3.5 Choice between, slab, girder, arch, cable-stayed or suspension bridges

Slab bridges are constructed for small spans ; girder, arch and truss bridges are constructed for medium to moderately large spans and the last mentioned two bridges viz. cable-stayed bridges and suspension bridges are constructed for large spans. Therefore, slab bridges are selected where bed scour is negligible and foundation cost is much less as in shallow raft foundations (Fig. 4.1). Choice of girder and truss bridges may be justified where deep foundations are required from scour and soil strata considerations but navigation clearance or free-board is comparatively less as in Mokamah Bridge (Fig. 4.12a) or Howrah Bridge (Fig.



(a) MOKAMAH BRIDGE OVER GANGA RIVER (BIHAR)



(b) CORONATION BRIDGE OVER TEESTA RIVER (WEST BENGAL)

FIG. 4·12

17.8). In the latter case, the free-board above the highest tide level is 8.84 metres. In a narrow gorge where good rock is available on both the banks, an arch bridge is the obvious choice (Fig. 4.12b). Arch bridges are unsuitable at locations where abutment foundations are susceptible to large movements both downwards and sideways.

Cable-stayed and suspension bridges are favoured where large clearance or free-board is required above H.F.L. or H.T.L. for the passage of large vessels. Free board of 34.78 metres has been proposed for the cable-stayed bridge now under construction at Calcutta (second Hooghly Bridge — Fig. 17.17). Free-boards of 36.6 metres, 46.2 metres and 69.5 metres have been provided for Severn Bridge (Fig. 17.25), Mackinac Bridge (Fig. 17.24) and Verrazano Narrows Bridge (Fig. 17.26) respectively. The channels being deep, construction of superstructure by staging or false-work is not possible neither the erection of the superstructure can be done by the normal erection procedure. In such cases, the cables are taken advantage of for the erection of the deck superstructure for both the cable-stayed and suspension bridges.

4.3.6 Choice of span for bridges

The following may be taken as rough guide for the selection of span lengths for bridges to give economical design.

a) For masonry arch bridges : S = 2H

b) For R. C. C. slab bridges : S = 1.5 H

Where S = Clear span in metres.

H = Total height of abuntment or pier from the bottom of foundation upto its top. For arch bridges, it is measured from foundation to the intrados of the keystone.

c) For medium to major bridges with deep foundation :

Cost of supporting system of the superstructure of one span = cost of one pier with its foundation.

This can be established theoretically with the following assumptions :

i) The bridge has equal spans, i.e., L = nS where L is the length of the bridge, n and S are number of spans and span length of the bridge.

ii) Cost of deck slab, wearing course, railing etc. varies as the span. If 'a' is the cost per metre of bridge, then cost per span = aS.

iii) Cost of supporting system i.e., main girder, cross girder etc. varies as the square of the span i.e. cost per span = bS^2 where b is a constant.

iv) Cost of one pier with its foundation is constant and is equal to P (Say).

v) Cost of abutment, wingwall etc. is constant and is equal to A (Say) for each side.

vi) Cost of each approach — B (Say).

Therefore, cost of the bridge $= n\,(aS + bS^2) + (n - 1)\,P + 2A + 2B$

Since, $n = \frac{L}{S}$, we may write

$$\text{Cost of the bridge, } C = La + LbS + \left(\frac{L}{S} - 1\right)P + 2A + 2B$$

$$= La + LbS + \frac{LP}{S} - P + 2A + 2B$$

For minimum cost of the bridge, $\frac{dC}{dS}$ must be eqal to zero, i.e. $Lb - \frac{LP}{S^2} = 0$ or, $b\,S^2 = P$. i.e., cost of supporting system of one span = Cost of one pier with its foundation.

It is needless to say that the above economic criterion does not hold good for small as well as for long span bridges. For small span bridges, the cost of superstructure becomes very much less whereas the same for the substructure including the abutments and wing walls is much more depending upon the smallness. On the otherhand, for long span bridges, the cost of the superstructures comes to few times the cost of substructure. Their choice is, therefore, depends upon some other factors as stated before rather than economics.

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CHAPTER 5

HIGHWAY BRIDGE LOADING, GENERAL FEATURES OF DESIGN AND PERMISSIBLE STRESSES

5.1 GENERAL

5.1.1 The following loads, forces and stresses shall have to be considered in the design of road bridges and all members shall be designed to sustain safely the effect of various loads, forces and stresses that may act together.

1. Dead load
2. Live load
3. Impact of the live load
4. Water current
5. Longitudinal forces caused by the tractive effort or braking effect of vehicles, F_b and/or those caused by the restraint to movement of free bearings, F_f
6. Centrifugal force
7. Buoyancy
8. Earth pressure including live load surcharge
9. Temperature effects
10. Deformation effects
11. Secondary effects
12. Wind load
13. Wave pressure
14. Impact due to floating bodies or vessels
15. Erection effects
16. Seismic force

5.1.2 Permissible increase in the working stresses under the combined effect of various loads, forces and effects

(a) When loads, forces and effects mentioned in item 1 to 8 act together, no increase in the working stress is permissible.

(b) The working stresses may be increased by 15 percent when the effects of items 9 to 11 are also added to (a) above.

(c) The working stresses may be increases by $33^1/_3$ percent for any of the following combination of loads and forces.

i) Loads and forces under items 1 to 13.

ii) Loads and forces under items 1 to 12 + 14.

iii) Loads and forces under items 1 + 4 + 7 + 8 + 12 + 15 + F_f.

(d) The working stresses may be increased by 50 percent for the load combinations as under :

i) Loads and forces under items 1 to 11 + Seismic effect + Wave pressure.

ii) Loads and forces under c(iii) + Seismic effect – Wind load.

5.1.3 Permissible increase in foundation pressure under various combination of loads, forces and effects

The maximum foundation pressure due to any combination of the loads, forces and effects given under items 1 to 11 shall remain within the safe bearing capacity of the subsoil.

When the effects of wind or seismic force are added to those of the loads, forces and effects under items 1 to 8, the maximum bearing pressure may be increased by 25 percent. Wind and seismic forces shall not be assumed to act simultaneously. The effect of the items 9 to 11 may be ignored in the design of foundations. No tension shall be permitted under foundations except on rock in which case either the tension side shall be adequately anchored or the foundation pressure is re-calculated on the reduced area of contact not less than eighty percent of the base area and the maximum pressure is kept within the permissible value.

5.2 DEAD LOAD

5.2.1 The unit weights of various materials shall be assumed in the design as shown in Table 5.1.

TABLE 5.1 : UNIT WEIGHT OF VARIOUS MATERIALS *(IRC)*[1]

Sl. No.	Materials	Unit weight in T/m^3
1	Brickwork in lime mortar	1.8
2	Brickwork in cement mortar	1.9
3	Brickwork in cement mortar (Pressed)	2.2
4	Stone masonry in lime mortar	2.4
5	Coursed rubble stone masonry in cement mortar	2.6
6	Cement concrete (Plain)	2.2
7	Cement concrete (reinforced)	2.4
8	Cement concrete (Plain with plums)	2.3

TABLE 5.1 : UNIT WEIGHT OF VARIOUS MATERIALS *(IRC)* [1] [Contd.]		
Sl. No.	**Materials**	**Unit weight in T/m^3**
9	Cement concrete (prestressed)	2.5
10	Lime concrete with brick aggregate	1.9
11	Asphalt concrete	2.2
12	Compacted earth	1.8
13	Sand (loose)	1.4
14	Sand (wet compressed)	1.9
15	Gravel	1.8
16	Macadam (binder premix)	2.2
17	Macadam (rolled)	2.6

5.3 LIVE LOAD

5.3.1 All the new road bridges in India shall be designed as per Indian Roads Congress loadings which consist of three classes of loading viz., I.R.C. class AA, I.R.C. class A and I.R.C. class B loading. For bridges to be built in certain municipal limits, industrial areas and on certain specified Highways, single lane of class AA or two lanes of class A whichever produces worse effect is to be considered. All other permanent bridges shall be designed with two lanes of class A loading while two lanes of class B loading is applicable to bridges in specified areas or to the temporary type of structures such as timber bridges etc. Where class 70-R is specified (Appendix A), it shall be used in place of I.R.C. class AA loading.

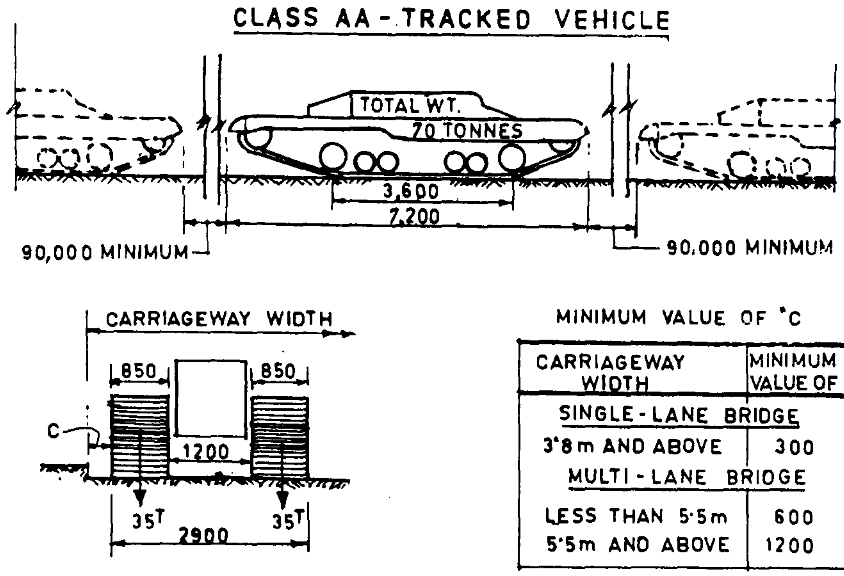
5.3.2 Fig. 5.1 and 5.2 show the I.R.C. loadings. These loads shall be assumed to travel along the longitudinal axis of the bridges and may be located anywhere on the deck for the consideration of worst effect produced in the section provided the distances between the wheel and the road kerb, the distances between axles or wheels and the distance between the adjacent vehicles as shown in the loading diagram are not encroached upon. Appendix A gives the essential data for the classification of bridges.

5.3.3 All the axles of a standard vehicle or train shall be considered as acting simultaneously and the space left uncovered by the standard train shall not be assumed as subject to any additional load. The trailers attached to the driving unit are not to be considered as detachable.

5.3.4 All new bridges shall be either one-lane, two-lane or four-lane width. Three-lane bridges shall not be considered. For four-lane bridges or multiple of two-lane bridges, at least 1.2 m wide central verge shall be provided.

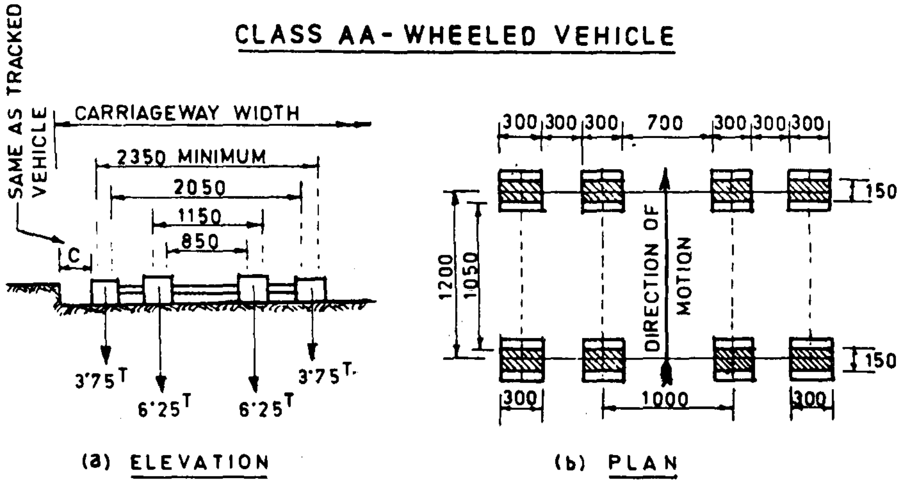
5.3.5 Reduction of stresses due to L.L. being on more than two traffic lanes simultaneously

The load intensity may be reduced by 10 percent for each additional traffic lanes in excess of the two lanes subject to a maximum reduction of 20 per cent and subject also to the condition that the load intensities as thus reduced are not less than the intensities resulting from a simultaneous loading on two lanes.



MINIMUM VALUE OF *C

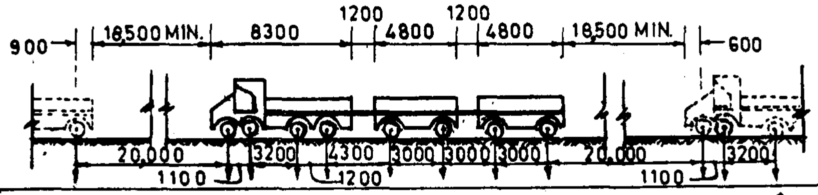
CARRIAGEWAY WIDTH	MINIMUM VALUE OF C
SINGLE-LANE BRIDGE	
3·8 m AND ABOVE	300
MULTI-LANE BRIDGE	
LESS THAN 5·5 m	600
5·5 m AND ABOVE	1200



NOTES FOR CLASS AA LOADING

1. The nose to tail spacing between two successive vehicles shall not be less than 90 m.
2. For multi-lane bridges, one train of class AA tracked or wheeled vehicles whichever creates severe conditions shall be considered for every two traffic lane width. No other live loads shall be considered on any part of the said two-lane wide carriageway of the bridge where above mentioned train of vehicle is crossing the bridge.
3. The maximum loads for the wheeled vehicle shall be 20 tonnes for a single axle or 40 tonnes for a bogie of two axles spaced not more than 1.2m c/c.
4. The minimum clearance between the road face of the kerb and the outer edge of the wheel or track, C, shall be as shown in the table.
5. All dimensions are in mm unless otherwise stated.

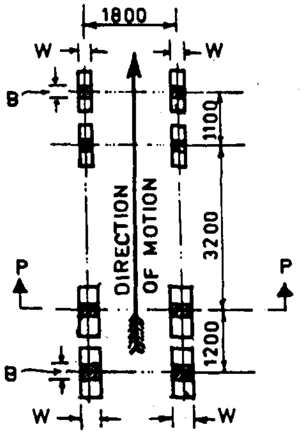
FIG. 5·1 IRC CLASS AA LOADING (IRC)[1]



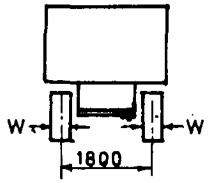
CLASS A	6.8	2.7	2.7	11.4	11.4	6.8	6.8	6.8	6.8	2.7	2.7	11.4
CLASS B	4.1	1.6	1.6	6.8	6.8	4.1	4.1	4.1	4.1	1.6	1.6	6.8

LOADS IN TONNES

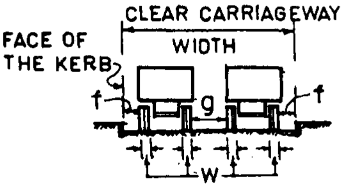
ELEVATION OF CLASS A OR CLASS B TRAIN OF VEHICLES



PLAN
OF DRIVING VEHICLE



SECTION - PP



TRANSVERSE DISPOSITION OF TWO TRAINS

VALUES OF 'B' AND 'W'

AXLE LOAD IN TONNES	GROUND CONTACT AREA B (mm)	GROUND CONTACT AREA W (mm)
CLASS A TRAINS		
11.4	250	500
6.8	200	380
2.7	150	200
CLASS B TRAINS		
6.8	200	380
4.1	150	300
11.6	125	175

VALUES OF 'g' AND 'f'

CLEAR CARRIAGEWAY WIDTH	g	f
5.5 TO 7.5 m	UNIFORMLY INCREASING FROM 400 TO 1200	150 mm FOR ALL CARRIAGEWAY WIDTH
ABOVE 7.5 m		

NOTES FOR CLASS A OR CLASS B LOADING
1. All dimensions are in mm unless otherwise stated.
2. The nose to tail distance between successive trains shall not be less than 18.5m.
3. No other live loads shall cover any part of the carriageway when a train of vehicles (or trains of vehicles in multi-lane bridge) is crossing the bridge.

FIG. 5.2 IRC CLASS A OR CLASS B LOADING (IRC)[1]

5.4 METHOD OF APPLICATION OF LIVE LOAD FOR THE DESIGN OF DECK SLAB

5.4.1 For slabs spanning in one direction only

A. ***Dispersion of load perpendicular to span***

(a) *Solid slab spanning in one direction*

i) For a single concentrated load, the effective width shall be calculated in accordance to the formula given hereunder. The effective width, however, shall not exceed the actual width of the slab.

$$b_e = Kx\left(1 - \frac{x}{L}\right) + W \tag{5.1}$$

Where b_e = the effective width of slab on which the load acts.

L = the effective span in case of simply supported span and the clear span in case of continuous span.

x = the distance of the C.G. of the concentrated load from the nearer support.

W = the dimension of the tyre contact area in a direction at right angles to the span plus twice the thickness of the wearing coat.

K = a coefficient having the values shown in Table 5.2 depending on the ratio of $\frac{b}{L}$ where b is the width of the slab.

TABLE 5.2 : VALUES OF COEFFICIENT, K, IN EQUATION 5.1 (IRC)[2]

$\frac{b}{L}$	K for simply supported slab	K for continuous slab	$\frac{b}{L}$	K for simply supported slab	K for continuous slab
0.1	0.40	0.40	1.1	2.60	2.28
0.2	0.80	0.80	1.2	2.64	2.36
0.3	1.16	1.16	1.3	2.72	2.40
0.4	1.48	1.44	1.4	2.80	2.44
0.5	1.72	1.68	1.5	2.84	2.48
0.6	1.96	1.84	1.6	2.88	2.52
0.7	2.12	1.96	1.7	2.92	2.56
0.8	2.24	2.08	1.8	2.96	2.60
0.9	2.36	2.16	1.9	3.00	2.60
1.0	2.48	2.24	2 & above	3.00	2.60

ii) For two or more concentrated loads in a line in the direction of the span, the bending moment per metre width shall be calculated separately for each load according to its appropriate effective width.

iii) For two or more loads across the span, if the effective width of slab for one load overlaps the effective width of slab for an adjacent load, the resultant effective width of slab for the two loads shall be taken as equal to the sum of the respective effective width for each load minus the width of overlap, provided that the slab is checked for the two loads acting separately.

(b) *Solid slab cantilever*

i) For a single concentrated load, the effective width of slab resisting the bending moment (measured parallel to the supported edge) shall be as follows :

$$b_e = 1.2\,x + W \tag{5.2}$$

Where b_e, x and W have the same meaning as before.

Provided that the effective width shall not exceed one-third the length of the cantilever slab measured parallel to the support and also provided that the effective width shall not exceed half the above value plus the distance of the concentrated load from the nearer extreme end when the concentrated load is placed near one of the two extreme ends of the cantilever slab.

ii) For two or more concentrated loads :

If the effective width of slab for one load overlaps the effective width for an adjacent load, the resultant effective width for two loads shall be taken as equal to the sum of the respective effective widths for each load minus the width of overlap provided that the slab so designed is tested for the two loads acting separately.

B. ***Dispersion of load along span***

The effective length of slab on which a wheel load or track load acts shall be taken as equal to the dimensions of the tyre contact area over the wearing surface of the slab in the direction of the span plus twice the overall depth of the slab inclusive of the thickness of the wearing coat.

5.4.2 For slabs spanning in two directions and for slabs spanning in one direction with width greater than 3 times the effective span

Adopt influence field, Piegeaud's or any other rational method with the value of Poisson's ratio as 0.15.

5.4.3 For ribbed slab or through slab other than solid slab

When the ratio of the transverse flexural rigidity to the longitudinal flexural rigidity is unity, the effective widths may be calculated as for solid slab. When the ratio is less than unity, a proportionately smaller value shall be taken.

5.4.4 Dispersion of loads through fills and wearing coat

The dispersion of loads through fills and wearing coat shall be taken at 45 degrees both along and perpendicular to the span.

5.5 FOOTWAY LOADING

5.5.1 For effective span of 7.5 m or less, 400 Kg/m^2. This load shall be increased to 500 Kg/m^2 for bridges near a town or centre of pilgrimage or large congregational fairs.

5.5.2 For effective span of over 7.5 m but not exceeding 30 m, the load intensity shall be calculated according to the following equation :

$$P = P' - \left[\frac{40L - 300}{9}\right] \tag{5.3}$$

5.5.3 For effective spans of over 30 m, the intensity of footway load shall be determined according to the following formula :

$$P = \left[P' - 260 + \frac{4800}{L}\right]\left[\frac{16.5 - W}{15}\right] \tag{5.4}$$

Where P' = 400 Kg/m^2 or 500 Kg/m^2 as the case may be
P = Footway load in Kg per m^2
L = Effective span of the main girder in metres
W = Width of footway in metres

5.5.4 The footway shall be designed to withstand load of 4 tonnes inclusive of impact distributed over an area having 300 mm diameter. In such case the permissible stresses may be increased by 25 percent to meet this provision. Where the vehicles cannot mount the footway, this provision need not be made.

5.6 IMPACT

5.6.1 Impact allowance as a percentage of the applied live loads shall be allowed for the dynamic action of the live loads as mentioned below :

5.6.2 For Class A or Class B Loading

Impact percentage shall be as shown in Fig. 5.3. Impact fraction shall be calculated from the following formulae for spans 3 m to 45 m.

(a) *For reinforced concrete bridges :*

$$\text{Impact fraction} = \frac{4.5}{6 + L} \tag{5.5}$$

(b) *For steel bridges :*

$$\text{Impact fraction} = \frac{9}{13.5 + L} \tag{5.6}$$

Where L = Length of span in metres as outlined in Art. 5.6.6.

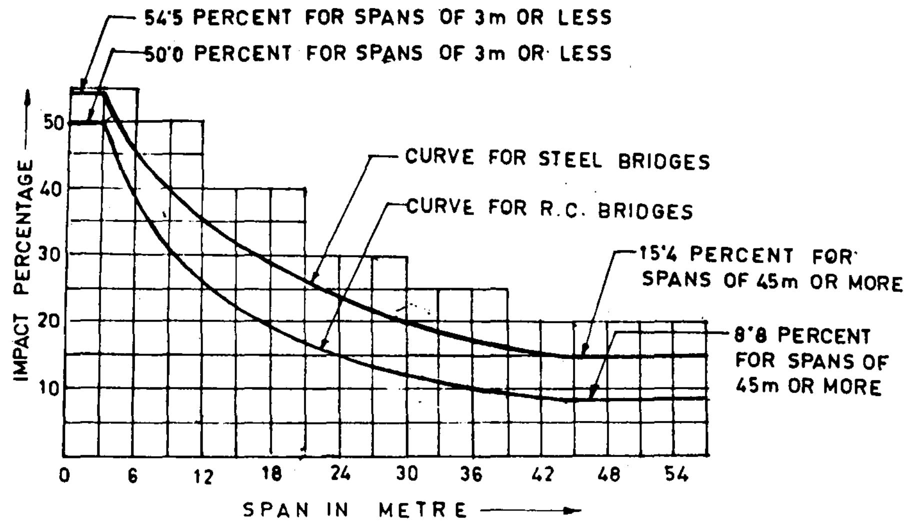


FIG. 5·3 IMPACT PERCENTAGE CURVES FOR HIGHWAY BRIDGES FOR CLASS A AND CLASS B LOADING (IRC)[1]

5.6.3 For class AA loading and Class 70R loading

The impact percentage shall be taken as mentioned below :

A. ***For span less than 9 m***

i) For tracked vehicles — 25 percent for spans upto 5 m linearly reducing to 10 percent for spans of 9 m.

ii) For wheeled vehicles — 25 percent.

B. ***For spans of 9 m or more***

(a) *Reinforced concrete bridges*

i) Tracked vehicles : 10 percent upto a span of 40 m and in accordance with the curve in Fig. 5.3 for spans in excess of 40 m.

ii) Wheeled vehicles : 25 percent for spans upto 12 m and in accordance with the curve in Fig. 5.3 for spans in excess of 12 m.

(b) *Steel bridges*

i) Tracked vehicles : 10 percent for all spans.

ii) Wheeled vehicles : 25 percent for spans upto 23 m and in accordance with the curve indicated in Fig. 5.3 for spans in excess of 23 m.

5.6.4 No impact allowance shall be allowed to the footway loading. For bridge structure having a filling not less than 600 mm including the road crust, the impact percentage shall be one- half of those specified in Art. 5.6.2 and 5.6.3.

5.6.5 The impact percentages at the followeing proportions shall be allowed for calculating the stresses at various points of piers and abutments from the top of bed block :

i) Pressure on the bearings and top surface of bed block	Full value
ii) Bottom surface of bed block	Half value
iii) From bottom surface of bed block upto 3 m of the structure below bed block	Half to zero decreasing uniformly
iv) 3 m below bottom of bed block	Zero

5.6.6 The span length, L, to be considered in the calculation of impact percentages as specified in Art. 5.6.2, and 5.6.3 shall be as under :

(a) For simply supported or continuous spans or for arches, L = the effective span on which the load is placed.

(b) For bridges having cantilever arms without suspended spans, L = the effective overhang of the cantilever reduced by 25 percent for loads on the cantilever arm and L = the effective span between supports for loads on the main span.

(c) For bridges having cantilever arms with suspended spans, L = the effective overhang of the cantilever arm plus half the length of the suspended span for loads on the cantilever arm and L = the effective length of the suspended span for loads on the suspended span and L = the effective span between supports for loads on the main span.

5.7 WIND LOAD

5.7.1 The wind load shall be assumed to act horizontally on any exposed portion of the bridge structure. The direction of the wind load may be such as to produce maximum resultant stresses in the member under consideration.

5.7.2 The wind force shall be assumed to act on the area of the structure as below :

(a) For deck structure — the area of the structure as seen in elevation including the floor system and railing less area of the perforation in the hand rails or parapet walls.

(b) For a through or a half-through structure — the area of the elevation of the windward truss as specified at (a) above plus half the area of elevation above the deck level of all other trusses or girders.

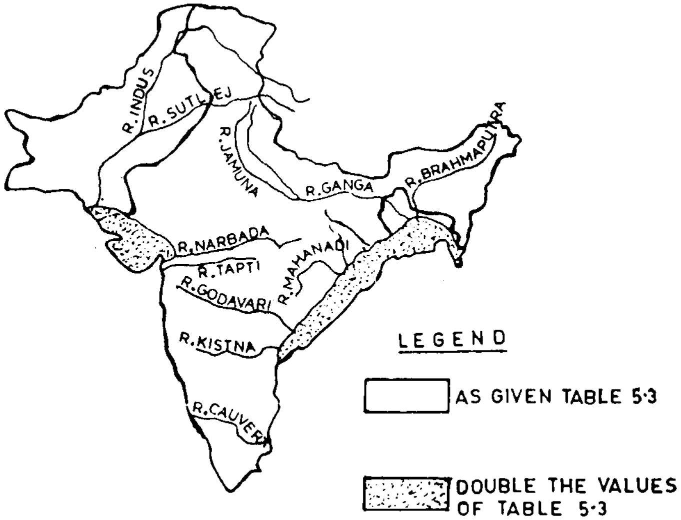


FIG. 5·4 INTENSITY OF WIND PRESSURE (IRC)[1]

5.7.3 The intensity of wind pressure shall be as per Table 5.3 below. The intensity may be doubled in certain coastal areas such as Kathiawar Peninsula, Bengal and Orissa coasts as shown in the map (Fig. 5.4).

No live load is assumed to travel through the bridge when the wind velocity at deck level exceeds 130 Km per hour.

TABLE 5.3 : WIND PRESSURES AT VARIOUS VELOCITIES *(IRC)*[1]

H	V	P	H	V	P
0	80	40	30	147	141
2	91	52	40	155	157
4	100	63	50	162	171
6	107	73	60	168	183
8	113	82	70	173	193
10	118	91	80	177	202
15	128	107	90	180	210
20	136	119	100	183	217
25	142	130	110	186	224

Where H = The average height in metres of the exposed surface above the mean retarding surface (ground or bed or water level).
V = Velocity of wind in Km per hour.
P = Intensity of wind pressure in Kg/m^2 at height H

5.7.4 The wind load on the moving live load shall be assumed to act at 1.5 m above the roadway at the rate of 300 Kg per linear metre of live load in case of ordinary bridges and 450 Kg per linear metre for bridges carrying tramway.

The total wind force shall not be less than 450 Kg per linear metre in the plane of the loaded chord and 225 Kg per linear metre in the unloaded chord on through or half-through truss, latticed or other similar spans, and not less than 450 Kg per linear metre on deck spans.

A wind pressure of 240 Kg per metre on the unloaded structure shall have also to be considered if it produces greater stresses than the wind loads mentioned previously.

5.8 HORIZONTAL FORCE DUE TO WATER CURRENTS

5.8.1 The effect of the horizontal force due to water currents shall have to be considered in designing any part of the bridge structure submerged in running water.

The intensity of water pressure due to water current may be calculated from the formula :

$$P = 52K\,U^2 \tag{5.7}$$

Where P = Intensity of pressure in Kg/m^2
U = the velocity of water current at the point under consideration in metre per second.
K = A constant having the values for different shapes of piers as shown in Table 5.4

TABLE 5.4 : VALUES OF CONSTANT, K, IN EQUATION 5.7 (IRC)[1]

Sl. No.	Shape of pier	Value of K
1	Square ended pier	1.50
2	Circular pier or pier with semi-circular ends.	0.66
3	Piers with cut and ease water at angle of 30 degrees or less.	0.50
4	Piers with cut and ease water at an angle of more than 30 degrees but less than 60 degrees.	0.50 to 0.70
5	Piers with cut and ease water at an angle of more than 60 degrees to 90 degrees.	0.70 to 0.90
6	Piers with cut and ease water of equilateral arcs of circle.	0.45
7	Piers with arcs of the cut and ease waters intersecting at an angle of 90 degrees.	0.50

5.8.2 The variation of U^2 may be assumed to be linear with zero value at the maximum scour level and the square of the maximum velocity at the surface (Fig. 5.5). The maximum surface velocity V_s may be taken as $V_m \sqrt{2}$, i.e. $V_s^2 = 2V_m^2$ where V_m is the mean velocity and may be determined from Art. 3.6.3. Therefore, U^2 in equation 5,7 at a depth X from the maximum scour level is given by —

$$U^2 = \frac{XVs^2}{H} = \frac{2XV_m^2}{H} \tag{5.8}$$

5.8.3 In order to provide against any possible variation of the direction of water-current from the normal direction of flow, provision may be made in the design by assuming a 20 degree inclination of the water-current with respect to the normal direction of flow. The velocity in such cases shall be resolved into two components viz. one parallel and the other normal to the pier. The values of K for normal component shall be taken as 1.5 except for circular piers when K may be taken as 0.66.

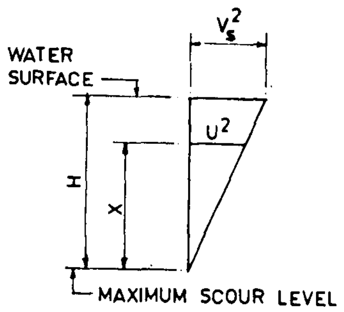


FIG. 5.5 WATER PRESSURE DIAGRAM (IRC)[1]

5.9 LONGITUDINAL FORCES

5.9.1 The effect of longitudinal forces due to tractive effort or braking effect (the latter being greater than the former) and the frictional resistance offered by the free bearing to movement due to change of temperature or any other cause shall have to be considered in the design of bearing, sub-structures and the foundations.

The horizontal force due to tractive or braking shall be assumed to act along the roadway and at 1.2 metres above it.

The braking and temperature effects on bridge structures having no bearings such as arches, rigid frames etc., shall be considered in accordance with the approved method of analysis of indeterminate structures.

5.9.2 For simply supported reinforced and prestressed concrete structures, plate bearings cannot be used for spans more than 15 metres.

5.9.3 For simply supported spans upto 10 metres where no bearings (except bitumen layer) are provided, horizontal force at the bearing level shall be :

$$\frac{F}{2} \text{ or } \mu\, Rg \text{ whichever is higher}$$

Where F = Applied horizontal force
μ = Coefficient of friction as given in Table 5.5.
Rg = Reaction due to dead load.

5.9.4 The longitudinal force at any free bearing (sliding or roller) for a simply supported bridge shall be taken as equal μR where μ is the co-efficient of friction and R is the sum of dead and live load reaction. The values of μ as shown in Table 5.5 are usually assumed in the design.

TABLE 5.5 : VALUES OF μ *(IRC)*[1]

Sl. No.	Type of bearing	Value of μ
1	Steel roller bearing	0.03
2	Concrete roller bearing	0.05
3	Sliding bearing — Teflon on stainless steel	0.05

TABLE 5.5 : VALUES OF μ *(IRC)*[1] [Contd.]		
Sl. No.	**Type of bearing**	**Value of μ**
4	Sliding bearing — stainless steel	0.15
5	Sliding bearing — grey cast iron on grey cast iron (mechanite)	0.40
6	Sliding bearing — steel on steel or steel on cast iron	0.50
7	Concrete over concrete with a bitumen layer in between	0.60

5.9.5 The longitudinal force at any fixed bearing for a simply supported bridge shall be as follows :

$$F - \mu R \quad \text{or,} \ \frac{F}{2} + \mu R \ \text{whichever is greater}$$

Where F = Applied horizontal force

μ = Coefficient of friction of free bearing as given in Table 5.5

R = Reaction due to dead and live loads.

5.9.6 Longitudinal force at each end of a simply supported structure having identical elastomeric bearings is given by $\frac{F}{2} + V_r\delta$ where V_r is the shear rating of the elastomeric bearing and δ is the movement of the deck due to temperature etc. other than due to applied forces.

5.9.7 The longitudinal forces on supports of a continuous structure shall be determined on the basis of the shear rating of the individual supports and the zero movement point of the deck.

5.9.8 The longitudinal and all other horizontal forces shall be calculated upto the level where the resultant passive earth pressure of the soil below the deepest scour level (or the floor level in case of a bridge having pucca floor) balances these forces.

5.9.9 The magnitude of the braking effect shall be assumed to have the following values :

i) For a single lane or two lanes bridge deck, braking effect shall be equal to twenty percent for the first train of vehicle plus ten percent for the succeeding trains or part thereof. Only one lane of train loads shall have to be considered in calculating the braking effect even when the bridge deck carries two lanes of train loads. The braking effect shall be equal to twenty percent of the load actually on the span where the entire first train is not on the span.

ii) For bridges having more than two lanes, braking effect shall be taken as equal to the value given in (i) above for two lanes plus five percent of the loads on the lanes in excess of two.

5.10 CENTRIFUGAL FORCES

5.10.1 For a curved bridge, the effect of the centrifugal force due to the movement of the vehicles in a curve shall be duly considered and the members have to be designed to cater for the extra stresses induced by the centrifugal action.

5.10.2 The centrifugal force shall be calculated according to the formula :

$$C = \frac{WV^2}{127\,R} \tag{5.8}$$

Where C	=	The centrifugal force in tonnes
W	=	Total live load in tonnes on the span
V	=	Design speed in Km per hour
R	=	Radius of curvature in metres

5.10.3 The centrifugal force shall be assumed to act at a height of 1.2 m above the roadway. No increase for impact effect shall be required. The centrifugal force shall be assumed to act at the point of action of the wheel loads or uniformly distributed over the length on which a uniformly distributed load acts.

5.11 BUOYANCY

5.11.1 The effect of buoyancy shall have to be considered in designing the members of the bridge structure if this consideration produces worst effect in the member. Due to buoyancy, a reduction in the weight of the structure is made.

5.11.2 If the foundation rests on homogeneous impervious strata, no provision for buoyancy effect is required to be made but if, on the otherhand, the foundation rests on pervious strata such as sand, silt etc., full buoyancy shall be considered. For other foundation conditions, including foundation on rock, some percentage of the full buoyancy shall be assumed as the buoyancy effect at the discretion of the bridge designer.

5.11.3 15 percent of full buoyancy shall be taken as the buoyancy effect for the submerged concrete or brick masonry structures due to pore pressure.

5.11.4 The effect of full buoyancy shall be duly considered in the design of superstructure for submersible bridges, if it produces greater stresses.

5.11.5 In case of deep foundations which displace water as well as soil mass such as sand, silt etc., the buoyancy causing reduction in weight shall be considered on two counts as under :

i) Buoyancy due to displaced water shall be taken as the weight of the volume of water displaced by the structure from the free surface of water upto the foundation level.

ii) Upward pressure due to submerged weight of soil calculated in accordance with Rankine's Theory.

5.12 EARTH PRESSURE

5.12.1 The earth pressure for which earth retaining structures are to be designed shall be calculated in accordance with any rational theory. Coulomb's earth pressure theory may be used subject to the modification that the resultant earth pressure shall be assumed to act at a height of 0.42H from the base, where H is the height of the retaining wall. The minimum intensity of horizontal earth pressure shall be assumed to be not less than the pressure exerted by a fluid weighing 480 Kg per cum.

5.12.2 All abutments shall be designed for a live load surcharge equivalent to 1.2 m height of earth fill. For the design of wing and return walls, the live load surcharge shall be taken as equivalent to 0.6 m height of earth fill.

5.12.3 The fills behind the abutments, wing and return walls which exert the earth pressure shall be composed of granular materials. A filter media of 600 mm thickness with smaller size towards the soil and bigger size towards the wall shall be provided over the entire surface of the abutments, wing or return walls.

5.12.4 Adequate number of weep holes shall be provided in the abutments, wing or return walls above the low water level for the drainage of accumulated water behind the walls. The spacing of the weep holes shall not exceed one metre in both horizontal and vertical directions. The size of the weep holes shall be adequate for proper drainage and the weep holes shall be placed at a slope towards outer face.

5.13 TEMPERATURE EFFECTS

5.13.1 All structures shall be designed to cater for the stresses resulting from the variation in temperature. The range of variation shall have to be judiciously fixed for the locality in which the structure is to be constructed. The lag between the air temperature and the interior temperature of massive concrete members shall be given due consideration.

5.13.2 The range of temperature as shown in Table 5.6 shall generally be assumed in the design.

5.13.3 The coefficient of expansion per degree celsius shall be taken as 11.7×10^{-6} for steel and R. C. structures and 10.8×10^{-6} for plain concrete structures.

TABLE 5.6 : TEMPERATURE RANGE TO BE CONSIDERED IN DESIGN *(IRC)* [1]

Type of structure	Moderate climate		Extreme climate		Remarks
	Temp. rise	Temp. fall	Temp. rise	Temp. fall	
Metal structure	50°C	18°C	50°C	35°C	Intermediate values can be allowed at the discretion of the designer
Concrete structure	17°C	17°C	25°C	25°C	

5.14 DEFORMATION EFFECTS (For steel-bridges only)

5.14.1 Deformation stress is caused by bending of any member of an open-web girder due to vertical deflection of the girder combined with the rigidity of the joints.

5.14.2 All steel bridges shall be designed, manufactured and erected in such a way that the deformation stresses are reduced to a minimum. In the absence of design calculations, deformation stresses shall not be less than 16 percent of the dead and live load stresses.

5.15 SECONDARY EFFECTS

5.15.1 Steel Structures

Seocondary stresses are addtional stresses caused by the eccentricity of connections, floor beam loads applied at intermediate points in a panel, lateral wind loads on the end posts of through trusses etc. and stresses due to the movement of supports.

5.15.2 Reinforced Concrete Structures

Secondary stresses are additional stresses caused by the movement of supports or by the deformation in the geometrical shape of the structure or restrictive shrinkage of concrete floor beams etc.

5.15.3 For reinformed concrete structures, the shrinkage coefficients shall be taken as 2×10^{-4}.

5.15.4 All bridges shall be designed and constructed in such a manner that the secondary stresses are reduced to a minimum.

5.16 WAVE PRESSURE

5.16.1 The wave forces shall be determined by suitable analysis considering drawing and inertia forces etc. on single structural members based on rational methods or model studies. In case of group of piles, piers etc. proximity effects shall also be considered.

5.17 IMPACT DUE TO FLOATING BODIES OR VESSELS

5.17.1 Members such as bridge piers, pile trestles etc. which are subject to impact forces of floating bodies or vessels shall be designed considering the effect of impact on such members. If the impact force strikes the members at an angle, the effect of the component forces shall also be duly considered.

5.18 ERECTION EFFECTS

5.18.1 The design office shall be supplied with the erection programme and the sequence of constructiron which the construction engineers desire to adopt and the designer shall account for in his design the stresses due to the erection effects. This shall include one span being completed and the adjacent span not in position.

5.19 SEISMIC FORCE

5.19.1 Fig. 5.6 shows the map of India indicating therein seismic Zone I to Zone V. All bridges in Zone V shall be designed for seismic forces as specified below. All major bridges with total lengths of more than 60 metres shall also be designed for seismic forces in Zone III and IV. Bridges in Zone I and II need not be designed for seismic forces.

5.19.2 The vertical seismic force shall be considered in the design of bridges to be built in Zone IV and V in which the stability is a criterion for design. The vertical seismic coefficient shall be taken as half of the horizontal seismic coefficient as given hereinunder.

5.19.3 When seismic effect is considered, the scour for the design of foundation shall be based on mean design flood. In the absence of detail data, the scour may be taken as 0.9 times the maximum scour depth.

5.19.4 Horizontal Seismic Force

5.19.4.1 The horizontal seismic force shall be determined by the followeing expression which shall be valid for bridges having span upto 150 m. In the case of long span bridges having spans greater than 150 m, design shall be based on dynamic approach.

$$F_{eq} = \alpha \cdot \beta \cdot \gamma \cdot G \tag{5.9}$$

Where F_{eq} = Seismic force

α = Horizontal seismic coefficient depending on location as given in Table 5.7 (for portion below scour depth, this may be taken as zero)

β = A coefficient depending upon the soil foundation system as given in Table 5.8.

α = A coefficient depending upon the importance of the bridge as given below. The importance shall be decided on local conditions such as strategic importance, vital communicatiron link etc.

(a) Important bridges 1.5

(b) Other bridges 1.0

G = Dead load or dead plus live load as the case may be vide Art. 5.19.4.3.

5.19.4.2 Horizontal seismic forces shall be taken to act at the centre of gravity of all loads under consideration. The direction of the seismic force shall be such that the resultant effect of the seismic force and other forces produces maximum stresses in the structure.

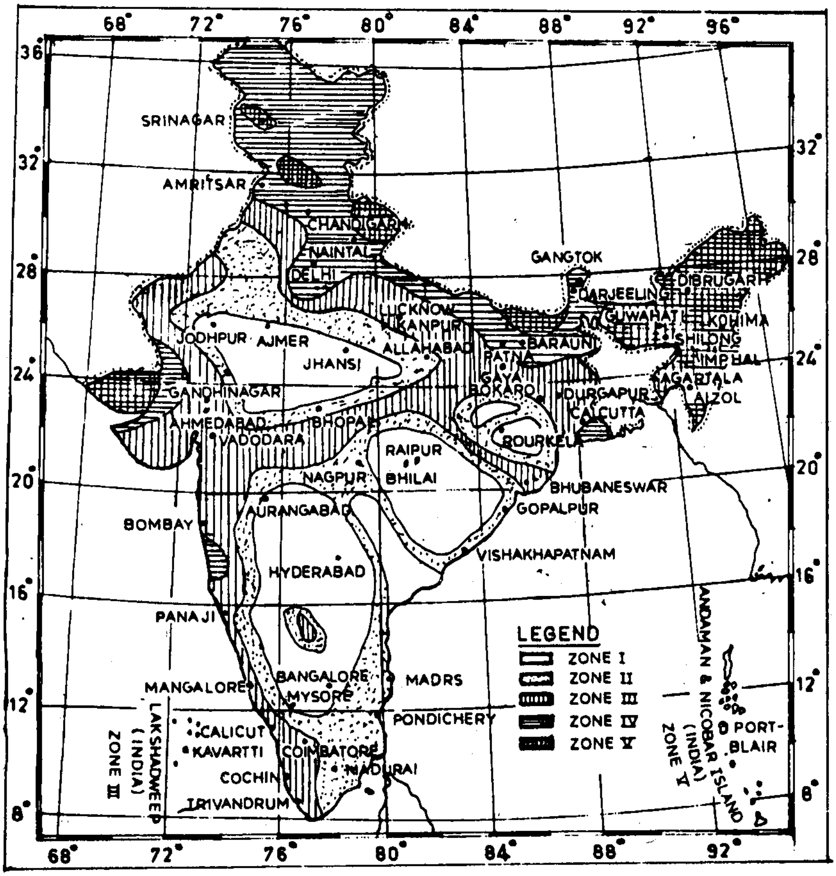


FIG. 5·6
MAP SHOWING THE SEISMIC ZONE OF INDIA (IRC)[1]

TABLE 5.7 : HORIZONTAL SEISMIC CO-EFFICIENT, α (IRC)[1]

Zone	Horizontal seismic coefficient, α
V	0.08
IV	0.05
III	0.04
II	0.02
I	0.01

5.19.4.3 The seismic force for live loads shall not be considered when acting in the direction of traffic but shall be considered in the direction perpendicular to traffic.

5.19.4.4 The portion of the structure embedded in soil shall not be considered to produce any seismic forces.

5.19.4.5 In loose or poorly graded sands with little or no fines, the vibrations due to seismic effect may cause liquifaction of soil or excessive total and differential settlement. Therefore, the founding of bridges on such strata in Zones III, IV and V shall be avoided unless appropriate methods of compaction or stabilisatiron are adopted.

5.19.4.6 Masonry or unreinforced concrete bridges shall not be constructed in Zone V.

TABLE 5.8 : VALUES OF β FOR DIFFERENT SOIL AND FOUNDATION SYSTEM *(IRC)* [1]

Type of soil mainly constituting the foundation	Value of β for			
	Bearing Piles resting on soil type I or raft foundations	Bearing Piles on soil type II and III, friction piles, combined or isolated RCC footings with beams	Isolated RCC footings without the beams or unreinforced strip foundations	Well foundations
Type I — Rock or hard soil. SPT value, N = 30 .	1.0	1.0	1.0	1.0
Type II — medium soils. SPT value N = 10 to 30	1.0	1.0	1.2	1.2
Type III — soft soil, SPT value N = 10	1.0	1.2	1.5	1.5

5.20 INFLUENCE LINE DIAGRAMS

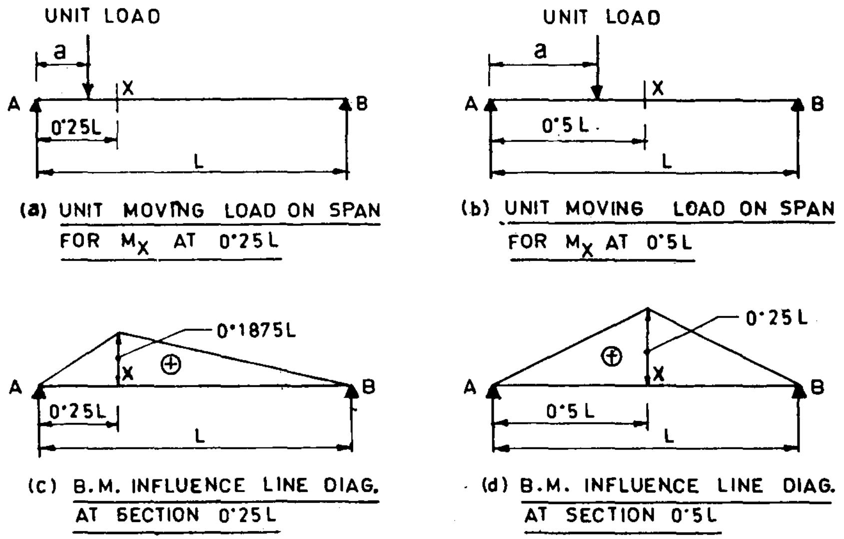
5.20.1 As stated in Art. 5.1.1 to 5.1.3, all structural members shall be designed with loads, forces and stresses that may act together. Most of these loads and forces have more or less fixed point of application except the live loads and the forces originating from live loads such as impact force, tractive or braking force and the centrifugal force. Since live loads are moving loads, their points of application have to be carefully determined in order to get maximum effect. This is achieved with the help of influence line diagrams as described in the paragraphs below.

5.20.2 An influence line is a curve which indicates the reaction, moment, shear, thrust etc. at a section of a beam or other members due to the movement of a unit concentrated load along the length of the beam or member. The procedure of drawing influence line diagram is illustrated in the following paragraphs. Influence line diagrams for some special structures such as R. C. continuous bridges and R. C. arch bridges have been shown in the respective chapters viz. in chapter 10 and chapter 13. The method of using these influence line diagrams for the determination of maximum values of moments, shears, reactions etc. have been indicated in various Illustrative Examples in the subsequent chapters viz. chapter 7, 8, 9, 11, 14 and 16.

5.20.3 Influence Line Diagram for Moment

5.20.3.1 *Simply supported Bridge-Section at 0.25L and 0.5L*

In Fig. 5.7(a), when a unit load is placed between A and X (i.e. the section under consideration), $R_B = \frac{a}{L}$ and $M_x = \frac{a \times 0.75L}{L}$ but when the unit load is between X and B, $R_A = \frac{(L-a)}{L}$, and $M_x = \frac{(L-a)\,0.25L}{L}$. The value of M_x will be maximum when the unit load is at X i.e. the section under consideration and the value of M_x = 0.1875L. The influence line diagram for M_x at 0.25L is shown in Fig. 5.7(c).



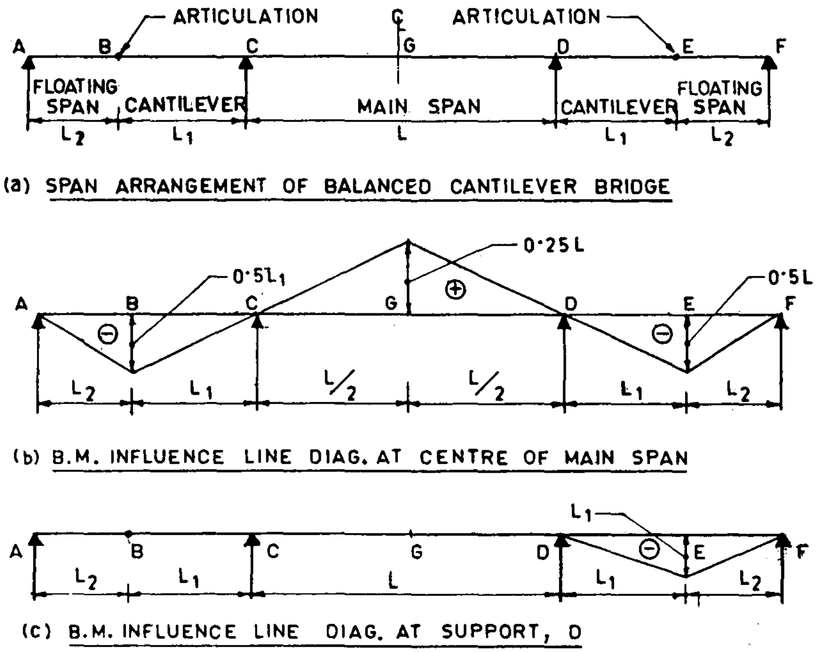
(a) UNIT MOVING LOAD ON SPAN FOR M_X AT 0·25 L

(b) UNIT MOVING LOAD ON SPAN FOR M_X AT 0·5 L

(c) B.M. INFLUENCE LINE DIAG. AT SECTION 0·25 L

(d) B.M. INFLUENCE LINE DIAG. AT SECTION 0·5 L

FIG. 5·7 B.M. INFL. LINE DIAGRAM AT SECTIONS 0·25 L AND 0·5 L FOR SIMPLY SUPPORTED BRIDGE



(a) SPAN ARRANGEMENT OF BALANCED CANTILEVER BRIDGE

(b) B.M. INFLUENCE LINE DIAG. AT CENTRE OF MAIN SPAN

(c) B.M. INFLUENCE LINE DIAG. AT SUPPORT, D

FIG. 5·8 B.M. INFL. LINE DIAGRAM FOR BALANCED CANTILEVER BRIDGE

Similarly, in Fig. 5.7(b), when the unit load is placed between A and X, $M_x = \frac{a \times 0.5L}{L}$ but when the unit load is placed between X and B, $M_x = \frac{(L-a) \times 0.5L}{L}$. The value of M_x is maximum when the unit load is placed at X in which case $M_x = 0.25L$. The influence line diagram for M_x at 0.5L is shown in Fig. 5.7(d).

5.20.3.2 *Balanced Cantilever Bridge — Section at centre of Main Span and at Support*

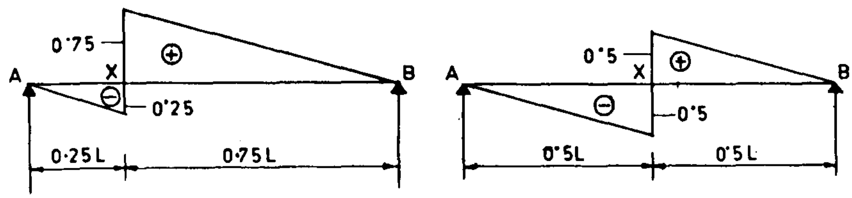
The influence line diagrams may be drawn in the same manner as illustrated in Art. 5.20.3.1. These diagrams are shown in Fig. 5.8.

5.20.4 Influence Line Diagram for Shear

5.20.4.1 *Simply supported Bridge — Section at 0.25L and 0.5L*

Referring to Fig. 5.7(a) when unit load is placed between A and X (i.e. the section under consideration), $R_B = \frac{a}{L}$ and S_x (i.e. shear at X) = $R_B = \frac{a}{L}$. As per normal convention, this shear i.e. resultant forces acting upwards on the right of section and acting downwards on the left of section is negative. When the unit load is between X and B, $R_A = \frac{(L-a)}{L}$ and S_x (shear at x) = $\frac{(L-a)}{L}$. This shear as per normal convention is positive. The shear changes sign when the unit load is at X. Therefore, the influence line diagram for shear at Section 0.25L will be as shown in Fig. 5.9(a). The ordinate of negative shear at X $= \frac{0.25L}{L} = 0.25$ and the ordinate of positive shear $= \frac{(L-0.25L)}{L} = 0.75$.

Referring to Fig. 5.7(b) it may be found as before that when the unit load is between A and X, $S_x = \frac{a}{L}$ and when the unit load is between X and B, $S_x = \frac{(L-a)}{L}$. The shear changes sign when the unit load is at the Section i.e., at 0.5L and the ordinates both for positive shear and negative shear are 0.5. The influence line diagram is shown in Fig. 5.9(b).



(a) SHEAR INFL. LINE DIAG. FOR SECTION X AT 0·25L

(b) SHEAR INFL. LINE DIAG. FOR SECTION X AT 0·5L

FIG. 5·9 INFLUENCE LINE DIAGRAM FOR SHEAR FOR SIMPLY SUPPORTED BRIDGE

5.20.4.2 *Balanced Cantilever Bridge — Section at centre of Main Span and at Support*

i) ***Section at centre of main span***

Referring to Fig. 5.8(a), when the unit load moves from A to G (i.e. the section under consideration), the reaction at D will be as follows :

Unit Load at	R_D
A	Zero
B	$\frac{L_1}{L}$ (downwards)
C	Zero
G	0.5 (upwards)

But when the unit load moves from G to F, the reaction at C will be as below :

Unit Load at	R_C
G	0.5 (upwards)
D	Zero
E	$\frac{L_1}{L}$ (downwards)
F	Zero

The reactions R_C or R_D is the shear at Section G. Using the normal sign convention, the influence line diagram for shear at Section G is as shown in Fig. 5.10(a).

ii) ***Section at Left of Support C***

Referring to Fig. 5.8(a), shear at left of Support C will be the load at C when the unit load moves from A to C and zero beyond C. Therefore, the shear influence line diagram will be as shown in Fig. 5.10(b).

iii) ***Section at Right of Support C***

Referring to Fig. 5.8(a), when the unit load moves from A to C the shear will be numerically equal to R_D and when the unit load moves beyond C, the shear will be numerically equal to R_C. The shear influence line diagram is shown in Fig. 5.10(c).

5.21 PERMISSIBLE STRESSES

5.21.1 Concrete Members

5.21.1.1 The permissible stresses for concrete of various grades shall be as shown in Table 5.9.

TABLE 5.9 : PROPERTIES AND BASIC PERMISSIBLE STRESSES OF CONCRETE *(IRC)*[2]

Sl. No.	Nature of properties/ stresses	Properties/Permissible Stress for concrete Grade					
		M 15	M 20	M 25	M 30	M 35	M 40
1.	Modulus of elasticity E_C, design value (GPa)	20	25	28	31	33	36
2.	Permissible direct compressive stresses (MPa)	3.8	5.0	6.2	7.5	8.5	8.5
3.	Permissible flexural compressive stresses (MPa)	5.0	6.7	9.3	10.0	11.5	11.5

Note : For calculating stresses in section, a modular ratio $\left(\frac{E_s}{E_c}\right)$ of 10 may be adopted.

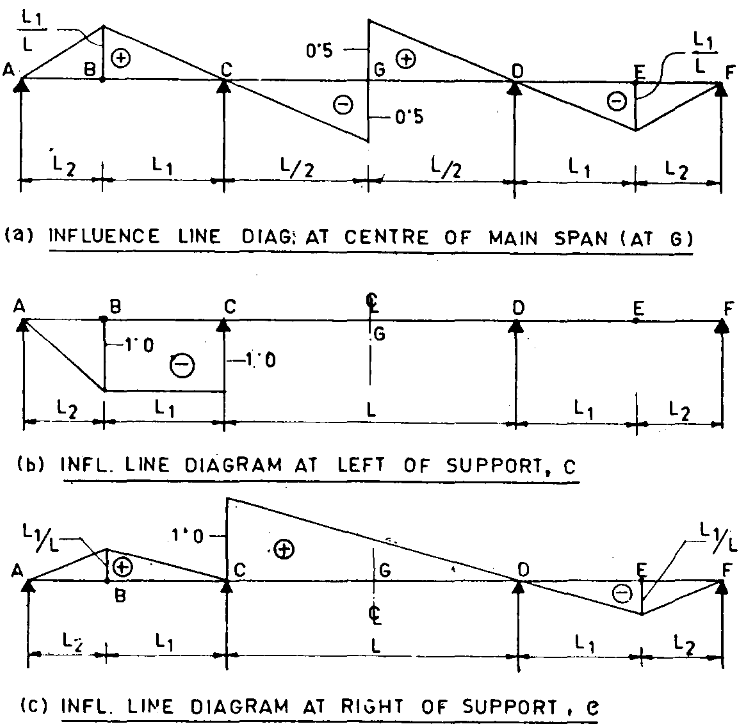


FIG. 5.10 INFLUENCE LINE DIAG. FOR SHEAR FOR BALANCED CANTILEVER BRIDGE

5.21.1.2 The permissible stresses in steel reinforcement shall be as indicated in Table 5.10.

TABLE 5.10 : PERMISSIBLE STRESSES IN STEEL REINFORCEMENT *(IRC)*[2]

Sl. No.	Type of stress in steel reinforcement	Bar Grade	Characteristic strength f_y (MPa)	Permissible stresses (MPa)
1.	Tension in flexure, shear or combined bending	S 240	240	125
		S 415	415	200
2.	Direct compression	S 240	240	115
		S 415	415	170

Note : The value of elastic modulus, E_s, may be taken as 200 GP, both for S 240 and S 415 grade.

5.21.1.3 The basic permissible tensile stresses in plain concrete shall be as given in Table 5.11

TABLE 5.11 : BASIC PERMISSIBLE TENSILE STRESSES IN PLAIN CONCRETE *(IRC)*[2]

Grade of Concrete	M15	M20	M25	M30	M35	M40
Permissible tensile stresses (MPa)	0.14	0.17	0.20	0.23	0.25	0.25

5.21.1.4 Reinforced concrete members may be designed without shear reinforcement if shear stress, $\tau \leq \tau_c$ where τ_c is given by the following expression :

$$\tau_c = K_1 K_2 \tau_{co}$$

Where τ_c = Permissible shear stress

K_1 = $[1.14 - 0.7\,d] \geq 0.5$; (d in m)

K_2 = $[0.5 + 0.25\rho] \geq 1$

τ_{co} = Basic values given in Table 5.12 for various grades of concrete

d = Effective depth of section

ρ = Longitudinal reinforcement ratio $\left(\frac{As}{bd}\right)$

As = Area of longitudinal reinforcement which continues at least, d, beyond the section considered or fully anchored when support section is considered.

b = Width of section or width of web in case of flanged beams.

TABLE 5.12 : BASIC τ_{co} VALUES FOR VARIOUS GRADES OF CONCRETE *(IRC)*[2]

Grade of Concrete	M15	M20	M25	M30	M35	M40
Values of τ_{co} (MPa)	0.28	0.34	0.40	0.45	0.50	0.50

5.21.1.5 The design shear stress $\tau = \frac{V}{bd}$ shall never exceed the maximum permissible shear stress τ_{max} as given below :

τ_{max} = 0.07 f_{ck} or 2.5 MPa whichever is less. Where f_{ck} is the characteristic strength of concrete.

5.21.2 Prestressed Concrete Members

5.21.2.1 *Grade of Concrete*

The characteristic compressive strength of concrete shall not be less than 35 MPa i.e. grade M 35 except for composite construction where concrete of grade M 30 could be permitted for deck slab.

5.21.2.2 *Permissible Temporary Stresses in Concrete*

These stresses are calculated after accounting for all losses except due to residual shrinkage and creep of concrete. The temporary compressive stress shall not exceed 0.5 f_{cj} which shall not be more than 20 MPa, where f_{cj} is the concrete strength at that time subject to a maximum value of f_{ck}.

At full transfer, the cube strength of concrete shall not be less than 0.8 f_{ck}. Temporary compressive stress in the extreme fibre of concrete (including stage prestressing) shall not exceed 0.45 f_{ck} subject to a maximum of 20 MPa.

Temporary tensile stress in the extreme fibre shall not exceed $^1/_{10}$th of the permissible temporary compressive stress in the concrete.

5.21.2.3 *Permissible Concrete Stresses during Service*

The compressive stress in concrete during service shall not exceed 0.33 f_{ck}. No tensile stress shall be permitted in the concrete during service.

If pre-cast segmental elements are joined by prestressing, the stresses in the extreme fibre of concrete during service shall always be compressive and the minimum compressive stress in an extreme fibre shall not be less than five percent of maximum permanent compressive stress that may be developed in the same section. This provision shall not, however, apply to cross-prestressed deck slab.

5.21.2.4 *Permissible Bearing Stress Behind Anchorages*

The maximum allowable stress immediately behind the anchorages in adequately reinforced end blocks may be calculated by the equation :

$$f_b = 0.48\, f_{cj} \sqrt{\frac{A_2}{A_1}} \text{ or } 0.8\, f_{cj} \text{ whichever is smaller}$$

Where f_b = the permissible compressive contact stress in concrete including any prevailing stress as in the case of intermediate anchorages.

A_1 = the bearing area of the anchorage converted in shape to a square of equivalent area

A_2 = the maximum area of the square that can be contained within the member without overlapping the corresponding area of adjacent anchorages and concentric with the bearing area A_1.

The above value of bearing stress is permissible only if there is a projection of concrete of at least 50 mm or $b_1/4$ whichever is more all round the anchorage, where b_1 is as shown in Fig. 5.11.

5.21.2.5 *Permissible Stresses in Prestressing Steel*

The maximum temporary stress in the prestressing steel at any section after allowing for losses due to slip of anchorages and elastic shortening shall not exceed 70 percent of the minimum ultimate tensile strength. Overstressing to compensate for slip of anchorages or to achieve calculated extension may be permitted subject to the jacking force limited to 80 percent of the minimum ultimate tensile strength or 95 percent of the proof stress (0.2 percent) of the prestressing steel whichever is less.

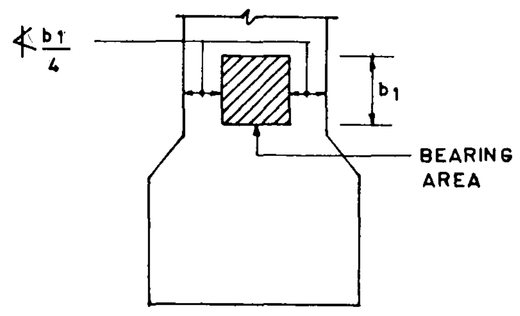


FIG. 5.11 (IRC)[4]

5.21.3 Structural Steel Members

Permissible stresses in structural steel members depend on many factors which have been dealt with in details in IRC : 24-1967 — Standard Specifications and Code of Practice for Road Bridges – Section V – Steel Bridges. For permissible stresses in steel section, the above code may be consulted.

5.22 REFERENCES

1. IRC : 6-1966 : Standard Specifications and Code of Practice for Road Bridges, Section II – Loads and Stresses (3rd Revision) – Indian Roads Congress.
2. IRC : 21-1987 : Standard Specifications and Code of Practice for Road Bridges, Section III; Cement Concrete (Plain and Reinforced) (2nd Revision), Indian Roads Congress.
3. IRC : 24-1967 : Standard Specifications and Code of Practice for Road Bridges, Section V – Steel Road Bridges, Indian Roads Congress.
4. IRC : 18-1985 : Design Criteria for Prestressed Concrete Road Bridges (Post-Tensioned Concrete) (2nd Revision) – Indian Roads Congress.

CHAPTER 6

LOAD DISTRIBUTION IN BRIDGE DECKS

6.1 GENERAL

6.1.1 When the loads are more or less evenly distributed over a stiff deck, the sharing of the loads between the girders is uniform and the load carried by any girder may be obtained if the total load is distributed over the girders evenly. The effect of a concentrated or a group of concentrated or partly distributed heavy load or loads in the bridge decks is somewhat different and the loads carried by different girders i.e. the distribution co-efficients, as are generally called, are different depending on the stiffness of the deck or the location of the load or loads on the deck. Since the longitudinal bending moments and shearing forces of the girders are dependent on the loads carried by them, these also vary from girder to girder and are not the same for all girders.

6.1.2 The following load distribution theories are generally adopted in distributing the live loads over the girders.

i) Courbon's theory
ii) Morice & Little's theory
iii) Hendry – Jaeger's theory

Of the above, the first one is the simplest in application but the results obtained therefrom are not so accurate except within certain range. The latter two are more rational and analytical. Morice & Little's theory is, however, found to give results which are realistic and show close agreement with test results as verified in a number of bridges from load tests. However, the former two load distribution methods will be described here.

6.2 COURBON'S THEORY

6.2.1 In Courbon's theory, the cross-beams or diaphragms are assumed to be infinitely stiff. Due to the rigidity of the deck, a concentrated load, instead of making the nearby girder or girders deflected, moves down all the girders the relative magnitude of which depends on the location of the concentrated load or group of concentrated loads. In case of a single concentric load or a group of symmetrical load, the deflection of all the girders becomes equal but when the loads are placed eccentrically with respect to the centre line of the deck, the deflection of all the girders does not remain the same but the outer girder of the loaded side becomes more deflected than the next interior girder and so on but the deflection profile remains in a straight line as illustrated in Fig. 6.1.

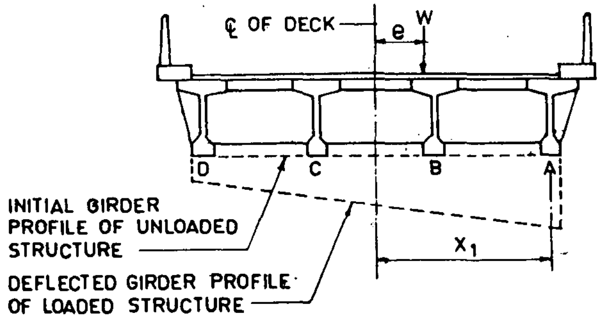


FIG. 6.1 DEFLECTION OF GIRDER UNDER ECCENTRIC LOAD

6.2.2 The behaviour of the deck is similar to a stiff pile-cap and the method of evaluation of load sharing or load distribution over the piles may be utilised in the evaluation of load coming on each girder.

Thus from Fig. 6.1

Load on beam A

$$= \left[\frac{W}{n} + \frac{Wex_1}{\Sigma x^2}\right] = W\left[\frac{1}{n} + \frac{ex_1}{\Sigma x^2}\right]$$

Distribution co-efficient, $K = \dfrac{\text{load carried by beam A}}{\text{Average load per beam}}$

$$= \frac{W\left[\dfrac{1}{n} + \dfrac{ex_1}{\Sigma x^2}\right]}{\dfrac{W}{n}} = \left[1 + \frac{nex_1}{\Sigma x^2}\right] \qquad (6.1)$$

When the girders have different moment of inertia, distribution co-efficient, K is given by,

$$K = \left[\frac{I_A}{I_a} + \frac{ne\,I_A x_1}{\Sigma Ix^2}\right] \qquad (6.2)$$

Where I_A = Moment of inertia of girder A

and I_a = Average moment of inertia of girders

6.2.3 Courbon's method is valid if the following conditions are satisfied :

i) The longitudinal girders are connected by at least five cross-girders, one at centre, two at ends and two at one-fourth points.

ii) The depth of the cross girder is at least 0.75 of the depth of the longitudinal girders.

iii) The span-width ratio is greater than 2 as specified in clause 305.9.1 of IRC:21-1987, (IRC Bridge Code Section III). The Author, however, recommends that to get realistic values, the span-width ratio shall be greater than 4 as was shown by the Author in an article published in the Indian Concrete Journal, August, 1965.

6.2.4 The use of Courbon's method in finding out the distribution co-efficients is illustrated by an example. It may be mentioned here that although the span-width ratio of the deck under consideration is not such as to make the theory valid but just to make a comparative study of the results by the other method viz. Morice and Little's theory, this is illustrated.

ILLUSTRATIVE EXAMPLE 6.1

Find out the distribution coefficients for the outer and central girder (having same moment of inertia) of the deck shown in Fig. 6.2 when single lane of class AA (tracked) loading is placed on the deck with maximum eccentricity. The distance between centre lines of bearings of the deck is 12 metres.

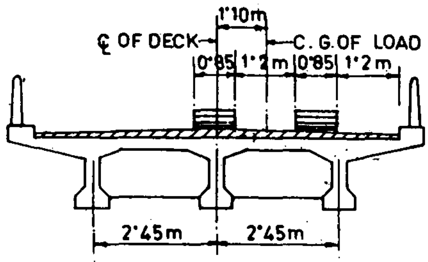


FIG. 6'2 CLASS AA TRACKED VEHICLE PLACED WITH MAXIMUM ECCENTRICITY (Example 6·1)

Solution

From equation 6.1,

Distribution coefficient on outer girder

$$= \left[1 + \frac{nex_1}{\Sigma x^2}\right] = \left[1 + \frac{3 \times 1.1 \times 2.45}{2 \times (2.45)^2}\right]$$

$$= \left[1 + \frac{8.085}{12.005}\right] = (1 + 0.67) = 1.67$$

Distribution coefficient on central girder

$$= \left[1 + \frac{3 + 1.1 \times 0}{2 \times (2.45)^2}\right] = (1 + 0) = 1.00$$

Live load moments on girders

Total live load moment at mid span.

= Area of the influence line diagram × intensity of load
$= 2 \times \frac{1}{2}(3.0 + 2.1) \times 1.8 \times 19.44 = 178.46$ tm
Impact factor = 10 percent
∴ Total L.L.M. with impact = 1.1 × 178.46 = 196.31 tm

Design L.L.M. on outer girder
= Average moment × distribution coeff.
$= \frac{196.31}{3} \times 1.67 = 109.28$ tm

Design L.L.M. on central girder $= \frac{196.31}{3} \times 1.0 = 65.44$ tm

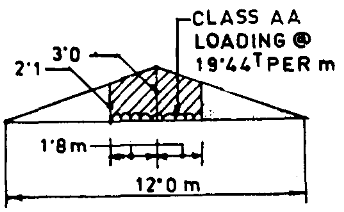


FIG. 6·3 INFLUENCE LINE DIAG. (Example 6·1)

6.3 MORICE & LITTLE'S THEORY

6.3.1 Unlike Courbon's theory, this theory takes into account the actual properties of the deck viz., the flexural and torsional stiffness of the deck and therefore, this method is considered to be more rational. The distribution coefficients obtained by this method fairly agree witht he actual load test results and therefore, the same is universally used.

6.3.2 In Morice & Little's theory, the properties of the deck have been expressed by the following two parameters :

$$\text{Flexural Parameter,} \quad \theta = \frac{b}{2a}\left[\frac{i}{j}\right]^{\frac{1}{4}} \quad (6.3)$$

$$\text{Torsional Parameter,} \quad \alpha = \frac{G(i_o + j_o)}{2E\sqrt{ij}} \quad (6.4)$$

6.3.3 The procedure for finding out the distribution coefficients by this method has been dealt with in detail in Morice & Cooley's book ***"Prestressed concrete — Theory and Practice"*** or Rowe's book ***"Concrete Bridge Design"*** and readers may, therefore, refer these books for the details. This is omitted for the fact that the Author's Simplified Method of Morice & Little's Theory has been explained in detail in Art. 6.4.

6.4 AUTHOR'S SIMPLIFIED METHOD OF MORICE & LITTLE'S THEORY

6.4.1 Though Morice and Little's method for finding out the distribution coefficients is more rational and gives better results, this method has at least one drawback in relation to Courbon's method viz. this method requires much more time in finding out the distribution coefficients. With a view to getting the distribution coefficients by the rational method of Morice & Little in comparatively lesser time, a simplified method (vide References) based on Morice & Little's theory has been developed by the Author. The principal feature of the simplified method is that instead of finding out the values of K_0 and K_1 from no- torsion and torsion graphs and then getting the value of K from the interpolation formula, $K = K_0 + (K_1 - K_0)\sqrt{\alpha}$, the value of K may be directly obtained from the curves (Fig. B-1 to B-9 in Appendix B) which have been prepared for various values of α and θ. The number of standard reference stations also has been reduced to five only viz., -b, -b/2, 0, b/2 and b instead of nine in order to keep the number of curves for the standard reference stations within practical limits.

6.4.2 The example used in finding out the distribution coefficients for the outer and central girders by the Courbon's method may again be tried by the simplified method of Morice & Little's Theory. This will explain the use of the simplified method for finding out the distribution coefficients as well as will assist in making a comparative study between the two methods.

ILLUSTRATIVE EXAMPLES 6.2

Calculate the distribution coefficients of the outer and central girder of the bridge deck shown in Example 6.1. Given :

i) Span = 2a = 12.0 m

ii) Nos. of main beams = m = 3

iii) Spacing of main beams = p = 2.45 m

iv) Equivalent width = 2b = mp = 3 × 2.45 = 7.35 m

v) Nos. of cross beams = 4

vi) Spacing of cross-beams = q = 4.0 m *vii) E = Young's Modulus = 35.25×10^4 Kg/cm^2*

viii) G = Rigidity Modulus = 14.10×10^4 Kg/cm^2

Solution

Moment of inertia of main beams :

Effective width of flange shall be minimum of the following values as per clause 305:12.2 of IRC:21-1987.

a) Spacing of girders = 2.45 m = 245 cm
b) 12 times the flange thickness plus rib width = $12 \times 23 + 30 = 306$ cm
c) $\frac{1}{4}$ Span = 3.0 m = 300 cm

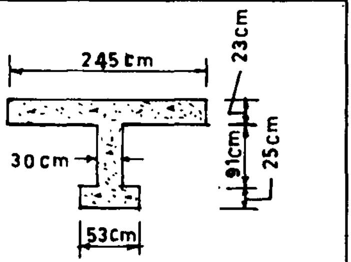


FIG. 6.4 IDEALISED SECTION OF MAIN BEAM (Ex.6-2)

For calculating the moment of inertia, an idealised section of the girder as shown in Fig. 6.4 is assumed. M.I. of main beam about centroid of section = 18.80×10^6 cm. units

$$i = \frac{I}{p} = \frac{18.80 \times 10^6}{245} = 7.67 \times 10^4 \text{ cm. units}$$

Torsional stiffness of main beam

$$I_0 \text{ (top flange)} = \frac{245 \times (23)^3}{6} = 49.68 \times 10^4 \text{ cm. units}$$

$$I_0 \text{ (bottom bulb)} = \frac{53 \times (25)^3}{6} = 13.80 \times 10^4 \text{ cm. units}$$

$$I_0 \text{ (web)} = \frac{91 \times (30)^3}{3} = 81.90 \times 10^4 \text{ cm. units.}$$

$$\therefore \text{ Total } I_0 = (49.68 + 13.80 + 81.90) \times 10^4 = 145.38 \times 10^4 \text{ cm. units.}$$

$$\therefore i_0 = \frac{I_0}{p} = \frac{145.38 \times 10^4}{245} = 0.59 \times 10^4 \text{ cm. units.}$$

Moment of inertia of the cross beam

Effective flange width shall be minimum of the followeing :

a) Spacing of cross beam = 4m = 400 cm.
b) 12 times the flange thickness plus rib width = $12 \times 23 + 25 = 301$ cm.
c) $\frac{1}{4}$ of span of the cross beam (assumed equal to the centre distance between outer girders)

$$= \frac{2 \times 245}{4} = 122.5 \text{ cm.}$$

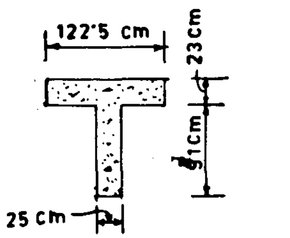


FIG. 6.5 SECTION OF CROSS-BEAM (Example 6.2)

Minimum value of 122.5 cm. is taken as the effective flange width.
Moment of inertia of the cross-beam, $J = 5.78 \times 10^6$ cm. units

$$\therefore j = \frac{J}{q} = \frac{5.78 \times 10^6}{400} = 1.45 \times 10^4 \text{ cm. units}$$

Torsional stiffness of the cross-beam

Effective flange width for cross beams may be taken as the spancing of the cross beam while finding out the torsional stiffness.

$$\therefore\ J_0 = \frac{400 \times (23)^3}{6} + \frac{91 \times (25)^3}{3}$$

$$= 81.11 \times 10^4 + 47.40 \times 10^4 = 128.51 \times 10^4 \text{ cm. units}$$

$$\therefore\ j_0 = \frac{J_0}{q} = \frac{128.51 \times 10^4}{400} = 0.32 \times 10^4 \text{ cm. units}$$

$$\therefore\ \theta = \frac{b}{2a}\left[\frac{i}{j}\right]^{\frac{1}{4}} = \frac{3.675}{12.0}\left[\frac{7.67 \times 10^4}{1.45 \times 10^4}\right]^{\frac{1}{4}} = 0.46, \text{ and}$$

$$\alpha = \frac{G\,(i_0 + j_0)}{2E\sqrt{ij}} = \frac{14.10 \times 10^4\,(0.59 \times 10^4 + 0.32 \times 10^4)}{2 \times 35.25 \times 10^4 \sqrt{7.67 \times 10^4 \times 1.45 \times 10^4}} = 0.054$$

Load on Equilivalent deck

Equilivalent deck width = 2b = np = 7.35 m. The tracked vehicle is placed on the equlivalent deck with the same eccentricity as shown in Fig. 6.2. The equilivalent loads at standard reference stations are calculated as simple reaction considering the distance between reference stations as simply supported spans and each track load as unit load.

TABLE 6.1 : EQUIVALENT LOADS AT STANDARD REFERENCE STATIONS

Load position	-b	-b/2	0	+b/2	+b	Remarks
Equivalent load, λ	0	0.04	0.89	0.91	0.16	$\Sigma\lambda = 2.0$

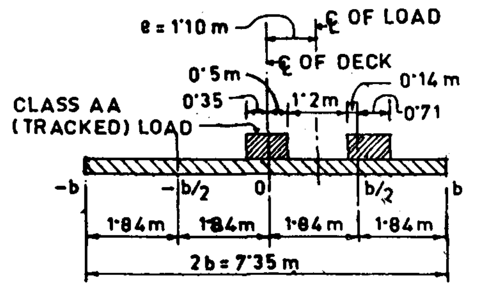


FIG. 6·6 (Example 6·2)

Unit Distribution Co-efficient, k

The unit distribution coefficients at various reference stations for equilvalent loads at various positions as in Table 6.1 are obtained from curves B-1 to B-9 (Appendix B) with $\theta = 0.46$ and $\alpha = 0.054$ and shown in Table 6.2.

TABLE 6.2 : UNIT DISTRIBUTION CO-EFFICIENTS

Load Position	Reference Stations				
	-b	-b/2	0	+b/2	+b
-b/2	2.20	1.70	1.00	0.32	-0.30
0	0.67	1.00	1.30	1.00	0.67
+b/2	-0.30	0.32	1.00	1.70	2.20
+b	-1.05	-0.30	0.67	2.10	3.94

Distribution Co-efficients at various Reference Stations

The distribution co-efficients at various reference stations may be obtained by multiplying the equivalent load λ with the unit distribution coefficients, k, adding vertically $\Sigma\, \lambda\, k$ and then dividing by 2 since there are two unit loads on the deck. In case of 2 lanes of class A loading, there will be four unit loads on the deck and as such $\Sigma\lambda k$ shall be divided by 4 to get distribution coefficients for each reference station.

TABLE 6.3 : DISTRIBUTION CO-EFFICIENTS AT VARIOUS STATIONS

Load position	Equivalent load, λ	λ k at reference station				
		-b	-b/2	0	b/2	b
-b/2	0.04	0.08	0.07	0.04	0.01	-0.01
0	0.89	0.60	0.89	1.16	0.89	0.60
b/2	0.91	-0.27	0.29	0.91	1.55	2.00
b	0.16	-0.17	-0.05	0.11	0.34	0.63
$\Sigma\lambda k$		0.24	1.20	2.22	2.79	3.22
$K = \frac{1}{2}\Sigma\lambda k$		0.12	0.60	1.11	1.40	1.61

Actual Distribution Co-efficients at Beam Position

Table 6.3 shows the distribution co-efficients at various reference stations but actual distribution co-efficients at beam positions are required to be known. This may be done by plotting the values of the distribution co-efficients at various reference stations on a graph paper wherein the beam positions are also shown. The distribution co-efficients may be read from the graph at the beams positions (Fig. 6.7). These values are shown in Table 6.4.

TABLE 6.4 : DISTRIBUTION CO-EFFICIENTS, K, FOR BEAMS

Beam No.	Beam 1	Beam 2	Beam 3	Remarks
K	0.44	1.11	1.45	$\Sigma k = 3.0$ i.e., the nos. of beams

6.4.3 It has been noted by comparison of the values of the distribution co-efficients obtained by Morice and Little's original method and by the Author's Simplified Method of Morice and Little's theory that results of both the methods are more or less the same and do not vary by more than 5 percent. Therefore, the simplified method presented herein may be adopted for practical design since this method is much quicker than the original method.

6.4.4 Live Load Moments on Girders

Total moment of the deck including impact as already worked out in Illustrative Example 6.1 is 196.31 tm.

$\therefore$ Design live load moment on outer girder = Average moment $\times$ distribution coefficient

$$= \frac{196.31}{3} \times 1.45 = 94.88 \text{ tm}$$

Design live load moment on central girder $= \dfrac{196.31}{3} \times 1.11 = 72.63$ tm

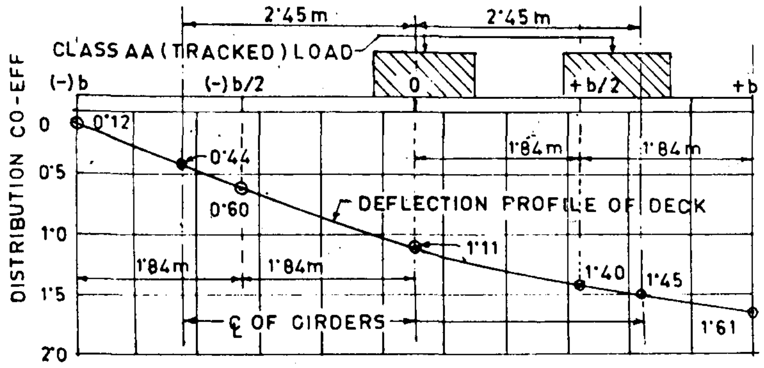


FIG. 6·7 DIST. COEFF. AT GIRDER POSITION (Ex. 6·2)

TABLE 6.5 : COMPARISON OF DISTRIBUTION CO-EFFICIENTS, K, BY VARIOUS METHODS

Value of	Courbon's method		Morice & Little's method	
	Central girder	Outer girder	Central girder	Outer girder
K	1.00	1.67	1.11	1.45

TABLE 6.6 : COMPARISON OF DESIGN LIVE LOAD MOMENTS ON LONGITUDINAL GIRDERS BY VARIOUS METHODS

Method	Design live load moment, tm		Remarks
	Central girder	Outer girder	
Courbon	65.44	109.28	Courbon's method gives 11 percent less moment for central girder and 13 percent more moment for outer girder in this particular case.
Morice and Little	72.63	94.88	

6.4.5 It is shown in Fig. 6.1 that the deflection profile of the main girder is assumed to be a straight line in Courbon's theory but in practice the transverse deck is not infinitely stiff although assumed in Courbon's theory. Morice and Little method, however, takes into consideration the actual properties of the transverse deck and as such the deflection profile is a curved one (concave in shape) as obtained in Fig. 6.7. This curved profile indicates that there is a transverse flexure in the bridge deck in addition to deflection of the longitudinal girders. Therefore, for realistic moments, Morice & Little's method shall be used. Where rough assessment is required within shortest possible time, Courbon's method may be adopted.

6.5 TRANSVERSE MOMENTS

6.5.1 So far the methods of distribution of live load on the longitudinal girders and therefore the procedures for finding out the bending moments on the longitudinal girders have been discussed. Now, the method of calculating the transverse moments and consequently the bending moments on the cross beams will be described. Each of the theories illustrated before for determining the distribution coefficient has its own method of finding out the transverse moments and will be discussed briefly in order to show the procedure for designing the cross beams of bridge decks.

6.5.2 Transverse moment by Courbon's method

Since the basic assumption of Courbon's theory is the infinite rigidity of the transverse deck, the moment in the transverse direction is found out by applying the same principle by which the moment in a stiff pile cap is determined. The loads transferred to the main beams are taken as the reactions of the supports.

6.5.3 Transverse moment by Morice & Little's method

The procedure for finding out the bending moment on the cross beam by Morice & Little's method has been described in details in Morice & Cooley's book and therefore, is not repeated here. Moreover, the Author's simplified method outlined hereafter which is based on Morice & Little's theory will tell about this method more or less in the same line.

6.5.4 Transverse moment by the Author's simplified method

When a load is placed on a bridge deck, it causes unequal deflection across a transverse sections and as such induces transverse bending moment.

This transverse bending moment is given by the infinite series :

$$M_y = \sum_{n=1}^{n=\alpha} \mu_{n\theta} \cdot r_n \cdot b \cdot \sin \frac{n\pi x}{2a} \tag{6.5}$$

It has been observed that first five terms are sufficient to get the moment at the centre of transverse span where the moment is maximum.

Therefore, equation 6.5 reduces to

$$M_y = b(\mu_\theta r_1 - \mu_{3\theta} r_3 + \mu_{5\theta} r_5)$$

Where μ_θ, $\mu_{3\theta}$, $\mu_{5\theta}$ are the transverse distribution coefficients for moments.

The value of θ is obtained from equation 6.3, i.e., from the structural properties of the deck. The term "r_n" is the n_{th} co-efficient of the Fourier Series representing the longitudinal disposition of the load (Fig. 6.8). The values of r_n for IRC class AA (tracked) or IRC class 70-R (tracked) and IRC class A or Class B loading are given below :

For Class AA or Class 70-R (tracked) loading

$$r_n = \frac{4w}{n\pi} \sin \frac{n\pi u}{2a} \sin \frac{n\pi c}{2a} \tag{6.7}$$

For moment at centre of span, where u = a (Fig. 6.9)

$$\therefore\ r_n = \frac{4w}{n\pi} \sin \frac{n\pi}{2} \sin \frac{n\pi c}{2a} \tag{6.8}$$

For Class A or B loading :

$$r_n = \frac{W}{a} \sin \frac{n\pi u}{2a} \tag{6.9}$$

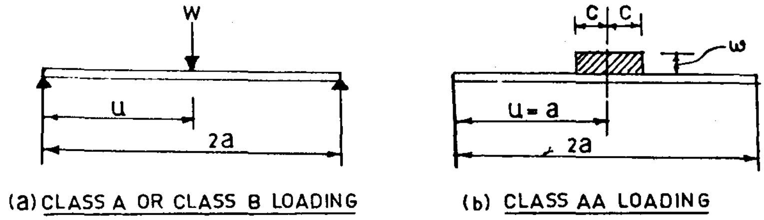


FIG. 6·8 LONGITUDINAL LOADING

The simplifications made in this method from the original method are :

i) values can be directly read from the curve instead of finding out the values of μ_0 and μ_1 from two set of curves and then getting μ values by applying the interpolation formula, $\mu = \mu_0 + (\mu_1 - \mu_0)\sqrt{\alpha}$ in each case.

ii) The value of $\sin\frac{n\pi u}{2a}$ and $\sin\frac{n\pi}{2}\ \sin\frac{n\pi c}{2a}$ can be determined from the curves B-13 to B-15 and the values of loading series r_n can be easily found out. The evaluatiron of these values otherwise takes considerable time.

The values of transverse co-efficients μ for variouys values of θ and α are shown in Fig. B-10 to B-12 (Appendix B) at the centre of the deck for load at (-)b, (-)b/2, 0, b/2 and b. The values of r_n for Class A or Class B, Class AA (tracked) and Class 70 R (tracked) loading can be easily determined from the curves as shown in Fig. B-13 to B-15 respectively (Appendix B).

ILLUSTRATIVE EXAMPLE 6.3

Find the design live load moment on the cross-beam of the bridge deck in example 6.1 by Courbon's method and Author's simplified Morice & Little's method.

Solution

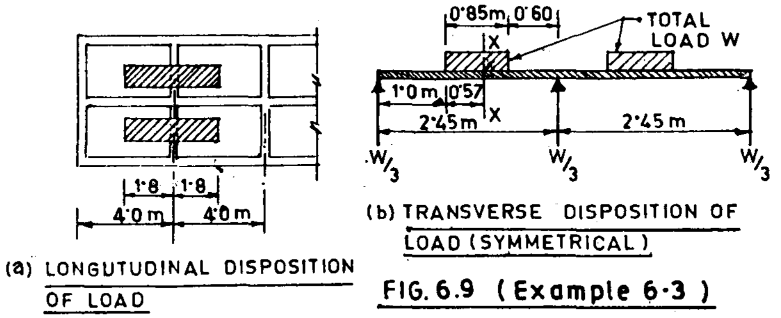


FIG. 6.9 (Example 6·3)

Courbon's method

i) Load placed symmetrically about centre line of transverse deck :

Considering longitudinal disposition (Fig. 6.9a), load transferred on the cross beam

$$= \frac{2 \times 35 \times 3.1}{4.0} = 54.25 \text{ tons} = \text{W (Say)}$$

Let the load W be placed symmetrically with respect to the C. L. of the deck as shown in Fig. 6.9b. Since the transverse deck is assumed to be rigid, the reaction on each longitudinal girder is $\frac{W}{3}$.

Now the moment on the cross beam will be maximum at the section where the shear is zero. This section is 1.57 m away from outer support (Section x-x).

$$\therefore M_{xx} = \frac{W}{3} \times 1.57 - \frac{W}{3} \times 0.285$$
$$= \frac{W}{3} \times 1.285 = \frac{54.25}{3} \times 1.285 = 23.24 \text{ tm}$$

M_{xx} with impact = 23.24 × 1.1 = 25.56 tm

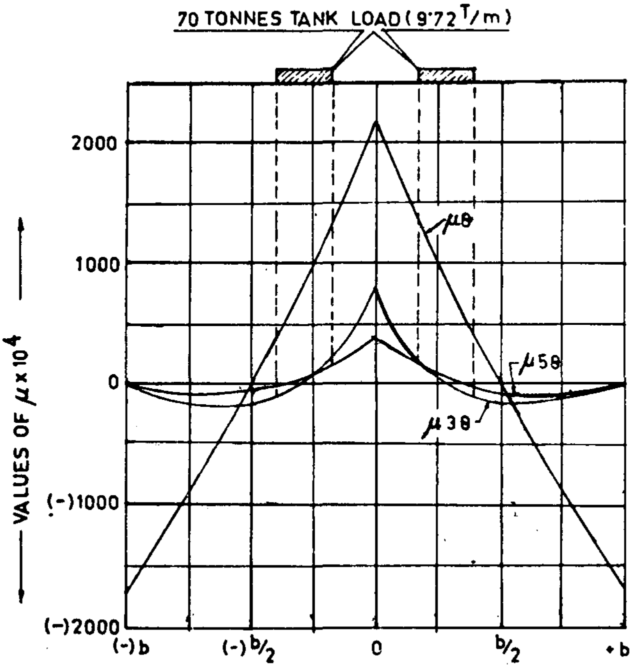


FIG. 6'10 INFLUENCE LINE FOR TRANSVERSE MOMENT CO-EFFICIENT (Ex. 6·3)

ii) Eccentric load on the deck :

It may also be examined if the bending moment produced on the cross beam due to eccentric load is more than that due to symmetrical load. The maximum of the two values shall have to adopted in the design.

Author's Simplified Morice & Little's Method

Symmetrical Load on Deck

The same deck as in example 6.1 is considered. The influence line diagrams for reference station, 0, i.e., at centre of deck (where the transverse moment will be maximum) are drawn for μ_0, μ_{30}, and μ_{50} with the values of $\theta = 0.46$ and $\alpha = 0.054$ as before and is shown is Fig. 6.10. Then after placing the tracks of Class AA loading on the influence line diagrams, the combined average ordinates of both the tracks are found which gives the values of μ_θ, $\mu_{3\theta}$ and $\mu_{5\theta}$ as 0.16, (-)0.020 & 0.020 respectively. Similarly, the value of $\sin\frac{n\pi}{2}\sin\frac{n\pi c}{2a}$ are obtained from Fig. B-14 (Appendix B) which are 0.48, (-)0.99 and 0.68 for n=1,3 and 5 respectively and for 2a = 12.0 m.

Transverse bending moment, per metre length, from equation 6.6

$$M_y = b\left[\mu_\theta r_1 - \mu_{3\theta} r_3 + \mu_{5\theta} r_5\right]$$

Substituting the values of r_n from equation 6.8.

$$M_y = \frac{4bw}{\pi}\left[\mu_\theta \sin\frac{\pi}{2}\sin\frac{\pi c}{2a} - \mu_{3\theta}\sin\frac{3\pi}{2}\sin\frac{3\pi c}{2a} + \mu_{5\theta}\sin\frac{5\pi}{2}\sin\frac{5\pi c}{2a}\right]$$

$$= \frac{4\times 3.675\times 9.72}{\pi}\left[0.16\times 0.48 + \frac{0.020}{3}\times(-)0.99 + \frac{0.020}{5}\times 0.68\right]$$

$$= 45.48\times 0.073 = 3.32 \text{ tm}$$

$\therefore$ Moment on the cross beam = Transverse moment per metre length $\times$ spacing of cross beam

$$= 3.32\times 4.0 = 13.28 \text{ tm}$$

With impact, moment = $1.1\times 13.28 = 14.61$ tm

This moment is required to be increased by 10 % due to local concentration of load.

$\therefore$ Nett live load moment on the cross beam including impact = $1.1\times 14.61 = 16.07$ tm

TABLE 6.7 : COMPARISON OF DESIGN LLM OF CROSS-GIRDER BY VARIOUS METHODS

Method	Design LLM	Remarks
Courbon	25.56 tm	Courbon's method gives 59 percent more moment than Morice & Little's method for the deck under consideration.
Morice & Little	16.07 tm	

6.5.5 Illustrative Example 6.2 and 6.3 showed the application of simplified Morice & Little's Method in respect of IRC class AA (Tracked) loads. This method may be used for IRC Class A or Class B loading also in the similar manner by placing the single lane or two lanes of vehicles as the case may be in the transverse direction with maximum eccentricity with respect to the centre line of the deck and calculating the equivalent loads at reference stations considering each wheel load as unit load. Therefore, $\Sigma\lambda$ must be equal to number of wheel loads, i.e., $\Sigma\lambda = 2$ for single lane loading and $\Sigma\lambda = 4$ for two lanes laoding. This implies that $K = \frac{1}{2}\Sigma\lambda k$ for single lane loading and $K = \frac{1}{4}\Sigma\lambda k$ for two lanes loading (Table 6.3). As regards longitudinal loading for the determination of transverse moments, the train loads shall be placed on the span to produce maximum moments and appropriate r_n values shall be used from equation 6.9. The wheel loads shall be placed symentrically with respect to the centre of the transverse deck.

6.5.6 As recommended in Art. 6.4.5, Morice & Little's method is more realistic and as such this method may be adopted in practical design for getting design moments. Where very rough and quick assessment of distribution coefficients is required, Courbon's method may be used.

TABLE 6.8 : MORICE'S DISTRIBUTION COEFFICIENTS FROM COURBON'S VALUES BY APPLYING MULTIPLYING FACTORS

Name of Bridge	Details of 7.5 m wide deck	Type of loading	Girder position	Courbon's K values before correction	Values of		Values of multi-plying factors from Fig. B-16 & B-17	Values of K obtained by		
					α	θ		Morice's Method	Cour bon's method after correc-tion	Load test
Road Over-bridge over G.T. Road, West Bengal	Span 27.44 m 7 beams @ 1.24 m	Two lanes of Class A	Central	1.00	0.023	0.28	1.03	1.03	1.03	–
			1st inner	1.14			1.01	1.15	1.15	–
			2nd inner	1.29			0.98	1.27	1.27	–
			Outer	1.43			0.946	1.35	1.35	–
Hindan Bridge at Gaziabad U.P.	Span 28.35 m 3 beams @ 3.20 m	Single lane of Class AA	Central	1.00	0.020	0.28	1.05	1.05	1.05	1.03
			Outer	1.46			0.925	1.35	1.35	1.35
Bhagirathi Bridge, West Bengal	Span 41.46 m 3 beams @ 1.48 m	Two lanes of Class A	Central	1.00	0.007	0.14	1.01	1.02	1.01	–
			Outer	1.32			0.97	1.26	1.28	–
Rupnarayan Bridge, West Bengal	Span 46.04 m 5 beams @ 1.88 m	Two lanes of Class A	Central	1.00	0.008	0.18	1.02	1.01	1.02	–
			Inner	1.15			0.987	1.12	1.13	–
			Outer	1.30			0.967	1.25	1.25	–
Ganga Bridge at Garhmuk-teswar, U.P.	Span 52.21 m 4 beams @ 1.90 m	Do	Inner	1.12	0.005	0.14	0.99	1.10	1.11	1.09
			Outer	1.35			0.975	1.30	1.31	1.30

6.6 MORICE'S DISTRIBUTION COEFFICIENTS FROM COURBON'S VALUES

6.6.1 As shown in Art. 6.2, Courbon's method of load distribution is very quick and simple but the distribution coefficients obtained by this method are not very realistic when the span-width ratio is less than 4. Morice's method of load distribution, however, gives correct results as verified by load tests in a number of bridges (Table 6.8). Therefore, it would be very advantageous if by some means the Morice's values of distribution coefficients are obtained by applying Courbon's theory.

6.6.2 Fig. B-16 & B-17 (Appendix B) give values of multiplying factors for certain values of α and θ, the parameters of the bridge deck. Morice's distribution coefficients may be obtained if Courbon's values are corrected by these multiplying factors. The correctness and usefulness of these multiplying factors in getting Morice's distribution co-efficients from Courbon's values within certain values of α and θ are shown in Table 6.8. These multiplying factors were developed by the Author and published in the Indian Concrete Journal — vide Reference 2.

6.7 REFERENCES

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2. Rakshit, K. S. — *"Load Distribution in Bridge Decks -- A Simplified Method"* — The Indian Concrete Journal, Bombay - 20 (June, 1964 and March & August, 1965 issues).
3. Rowe, R. E. — *"Concrete Bridge Design"* — C. R. Books Ltd., London
4. Sharan, shitla and Bhargava, J. K. — *"Load Test of Ganga Bridge at Garhmukteswar"* — Journal of the Indian Roads Congress, Volume – XXVII – Part I, Paper No. 236.

CHAPTER 7

SOLID SLAB BRIDGES

7.1 GENERAL

7.1.1 Simply supported solid-slab decks are adopted for culverts and for minor bridges of spans not exceeding 9 metres. Such type of decks have the following advantages over other type of superstructures.

i) Formwork is simpler and less costly.

ii) Smaller thickness of deck thereby reducing the height of fill and consequently the cost of the approaches.

iii) Simpler arrangement of reinforcement. No stirrups or web reinforcement are required. Reinforcement are evenly distributed throughout the full width of deck instead of being concentrated at girder points.

iv) Placing of concrete in solid slab is much easier than in slab and girder or any other similar type of bridges.

v) Chances of honey-combing in concrete are less.

vi) Cost of surface finish is less than girder bridges.

vii) Quicker construction.

7.1.2 The main disadvantages of solid slab bridges except for shorter spans are :

i) Greater cost of materials.

ii) Larger dead loads.

7.2 The design principles of a solid slab bridge deck may be illustrated by the following illustrative example.

ILLUSTRATIVE EXAMPLE 7.1

Design a solid slab bridge superstructure having a clear span of 9.0 metres and carriageway of 7.5 metres with 1.5 metres wide footway on either side for a National Highway. Loading : Single lane of IRC Class 70-R (both wheeled and tracked) or two lanes of IRC Class A whichever produces maximum effect.

Effective Span

Assume an overall depth of slab, D = 675 mm. and clear cover 30 mm.

∴ Effective depth, d = 675 – cover – half dia of bar = 675 – 30 – 13 = 632 mm.

∴ Effective span = clear span + effective depth
= 9.0 + 0.63 = 9.63 m

Dead Load

Load for per metre run per one metre width of slab is considered :

a)	675 mm. slab @ 2400 kg/m^3		1620 Kg.
b)	85 mm. (average) wearing course @ 2500 kg/m3		212 Kg.
c)	Extra load for wheel guard, parapet etc. (See Fig. 7.1)		
i)	Wheel guard = 2 × 0.28 × 0.225 × 1.0 × 2400	302 Kg.	
ii)	Support of parapet kerb = 2 × 0.28 × 0.29 × 1.0 × 2400	390 Kg.	
iii)	Parapet Kerb = 2 × 0.28 × 0.122 × 1.0 × 2400	164 Kg.	
iv)	Footway slab = 2 × 1.35 × 0.06 × 1.0 × 2400	389 Kg.	
v)	Railing = 2 × 150 Kgs.	300 Kg.	
	Total load (c)	1545 Kg.	

∴ Load per metre width of deck

$= \frac{1545}{11.03} = 140$ Kg. for wheel guard, parapet etc.

∴ Total dead load = (a) + (b) + (c)
= 1620 + 212 + 140 = 1972 Kg. per metre width

Dead load moment at mid span per metre width $= \frac{1972\,(9.63)^2}{8} = 22{,}860$ Kgm

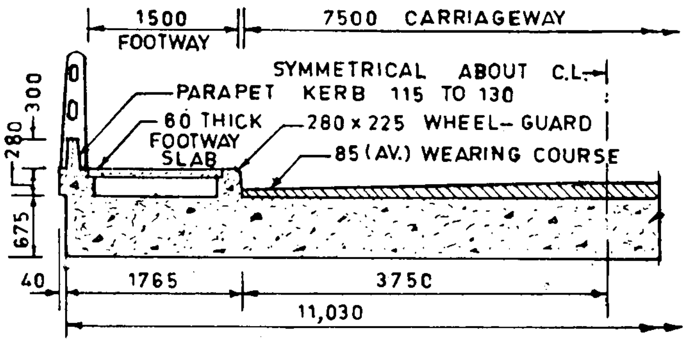

FIG. 7·1 CROSS-SECTION OF SOLID SLAB DECK

Live load moments :

The width is less than 3 times the effective span i.e. 11.03 m. < 3 × 9.63 (= 27.89 m) vide Art 5.4. Single lane of IRC 70-R tracked vehicle when placed at the centre will produce maximum moment. Two lanes of Class A loading or single lane of Class 70-R (wheeled vehicle) will not produce maximum moment.

Dispersion of load across span :

Effective width for a single concentrated load (Art. 5.4).

$$b_e = Kx\left(1 - \frac{x}{L}\right) + W \; ; \quad \frac{b}{L} = \frac{11.03}{9.63} = 1.15$$

∴ K for simply supported slab from Table 5.2 = 2.62 for $\frac{b}{L} = 1.15$; W = 0.84 + 2 × 0.085 = 1.01 m.

$\therefore\ b_e = 2.62 \times 4.815\left(1 - \dfrac{4.815}{9.63}\right) + 1.01.$

$= 2.62 \times 4.815 \times 0.5 + 1.01 = 7.32\text{m}.$

Therefore, the effective widths of both the tracks overlap (Fig. 7.2). When the tracked vehicle moves nearest to the road kerb —

$b_e = 3.66 + 2.04 + 3.385 = 9.085$ m.

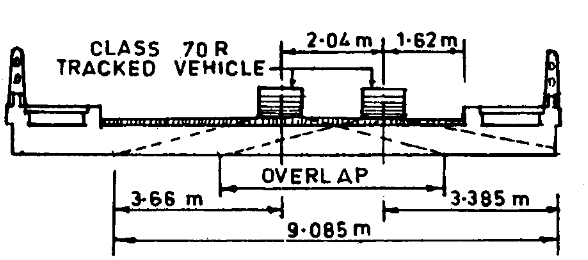

FIG. 7·2 DISPERSION OF WHEEL LOAD

Dispersion of load along span (vide Art. 5.4)

$= 4.57 + 2\,(0.675 + 0.085) = 4.57 + 1.47 = 6.09$ m.

$\therefore$ Intensity of live load $= \dfrac{70 \times 1000}{9.085 \times 6.09} = 1265\ \text{Kg/m}^2.$

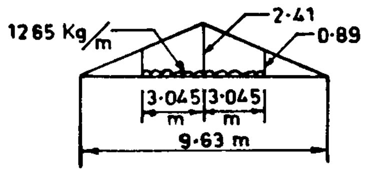

FIG. 7·3 INFLUENCE LINE DIAG. FOR MOMENT

Live load moment
= Area of influence line diagram × intensity of load

$= 2\,\dfrac{(0.89 + 2.41)}{2} \times 3.045 \times 1265$

= 12,700 Kgm. per metre width.

Impact factor from Art. 5.6 is 10 percent.

$\therefore$ Live load moment with impact $= 1.10 \times 12{,}700 = 13{,}960$ Kgm. per metre width

Footway loading :

From Art. 5.5, $P = P' - \dfrac{(40L - 300)}{9} = 400 - \dfrac{(40 \times 9.63 - 300)}{9} = 400 - 9 = 391\ \text{Kg./m}^2$

Total load on the bridge for two footpaths $= 2 \times 1.5 \times 391 = 1173$ Kg. per metre

$\therefore$ Footway loading per metre width of slab $= \dfrac{1173}{11.03} = 106$ Kg./m.

$\therefore$ Moment at midspan $= \dfrac{106 \times (9.63)^2}{8} = 1224$ Kgm.

Design moment = DLM + LLM + Footway loading moment = 22,860 + 13,960 + 1224 = 38,044 Kgm.
$= 38{,}044 \times 9.8 = 3{,}72{,}800$ Nm.

Design of Section :

M 20 grade concrete and HYSD bars (S 415) are proposed to be used in the slab. Therefore, the following design parameters are used in the determination of depth and reinforcement of the slab.

From Art. 5.21, $\sigma_c = 6.70$ MPa; $\sigma_s = 200$ MPa

From Art. 6.1 of *"Design Aids for Reinforced concrete to IS : 456 – 1978"*, the depth of neutral axis, lever arm factor, modular ratio etc. are determined as follows :

$$m = \frac{280}{3\,\sigma_c} = \frac{280}{3 \times 6.70} = 13.93$$

Neutral axis factor,

$$K = \frac{93.33}{6_s + 93.33} = \frac{93.33}{200 + 93.33} = 0.318$$

$\therefore$ Depth of neutral axis = 0.318 × 610 = 194 mm.

$$\therefore \text{ Lever arm, } jd = \left(d - \frac{0.318}{3} d\right) = 0.894\ d$$

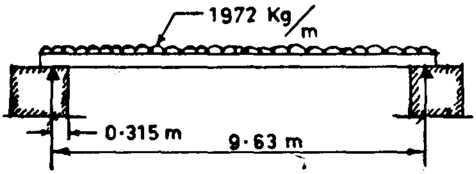

FIG. 7·4 DEAD LOAD SHEAR

$$\text{Moment of resistance} = \tfrac{1}{2} \cdot 6_c \cdot Kd \cdot b \cdot jd$$
$$= \tfrac{1}{2}\ 6.70 \times 0.318\ d \times b \times 0.894\ d = 0.95\ bd^2$$

$$\text{Effective depth, } d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{3,72,800 \times 10^3}{0.95 \times 10^3}} = 626 \text{ mm.}$$

Effective depth provided,

d = 675 - 30 (Cover) - 13 (half dia of bar) = 632 mm. Hence depth is safe.

Area of main reinforcement :

$$\text{Area of main reinforcement} = \frac{3,72,800 \times 10^3}{200 \times 0.894 \times 632} = 3299 \text{ mm}^2$$

25 Φ (i.e. HYSD bars) may be used at 140 mm. centres thereby providing a steel area of $\frac{1000}{140} \times 490 = 3500$ mm^2.

Area of Distribution Steel :

As per clause 305.15 of IRC bridge code Section III (IRC 21-1987),

Moment in the transverse direction = 0.3 × LLM + 0.2 × other moments

$$= 0.3 \times 13,960 + 0.2\ (22,860 + 1224)$$
$$= 4188 + 4817 = 9005 \text{ Kgm.} = 88,250 \text{ Nm}$$

Using HYSD bars also in the transverse direction. Area of distribution steel required

$$= \frac{88,250 \times 10^3}{200 \times 0.894\ (632 - 20)} = 806 \text{ mm}^2$$

Use 12 Φ distribution bars at 125 mm. centres thus giving an area of $\frac{1000}{125} \times 113 = 904$ mm^2

Shear Stress :

$$\text{Dead load shear} = 1972 \times \frac{9.63}{2} - 1972 \times 0.315 = 9495 - 622 = 8873 \text{ Kg/Metre width}$$

Live load shear :

For getting maximum L.L. shear, the C.G. of the tracked vehicle must be at a distance of half the longitudinal dispersion width i.e. $\frac{1}{2} \times 6.04$ m. = 3.02 m. Although dispersion width along span will remain unchanged, dispersion across span will vary.

Dispersion width across span from equation 5.1

$$be = Kx\left(1-\frac{x}{L}\right)+W = 2.62\times 3.04\left(1-\frac{3.04}{9.63}\right)+1.01 = 6.46\text{m.}$$

Taking the effect of two tracks, total dispersion width across span = 6.46 + 2.04 = 8.5 m.

$\therefore$ Intensity of load $= \dfrac{70\times 1000}{6.09\times 8.5} = 1352$ Kg./m^2

Live load shear = Area of the influence line diag. × intensity of load
$= \frac{1}{2}(1.00+0.37)\times 6.09\times 1352$
= 5640 Kg per Metre width.

Live load shear with impact = 1.1 × 5640
= 6050 Kg. per Metre width.

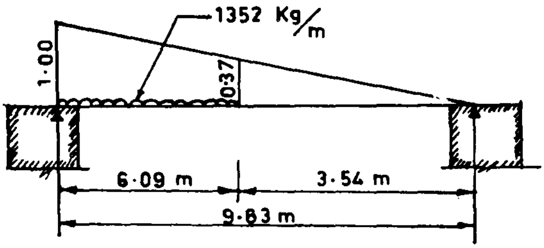

FIG. 7·5 LIVE LOAD SHEAR

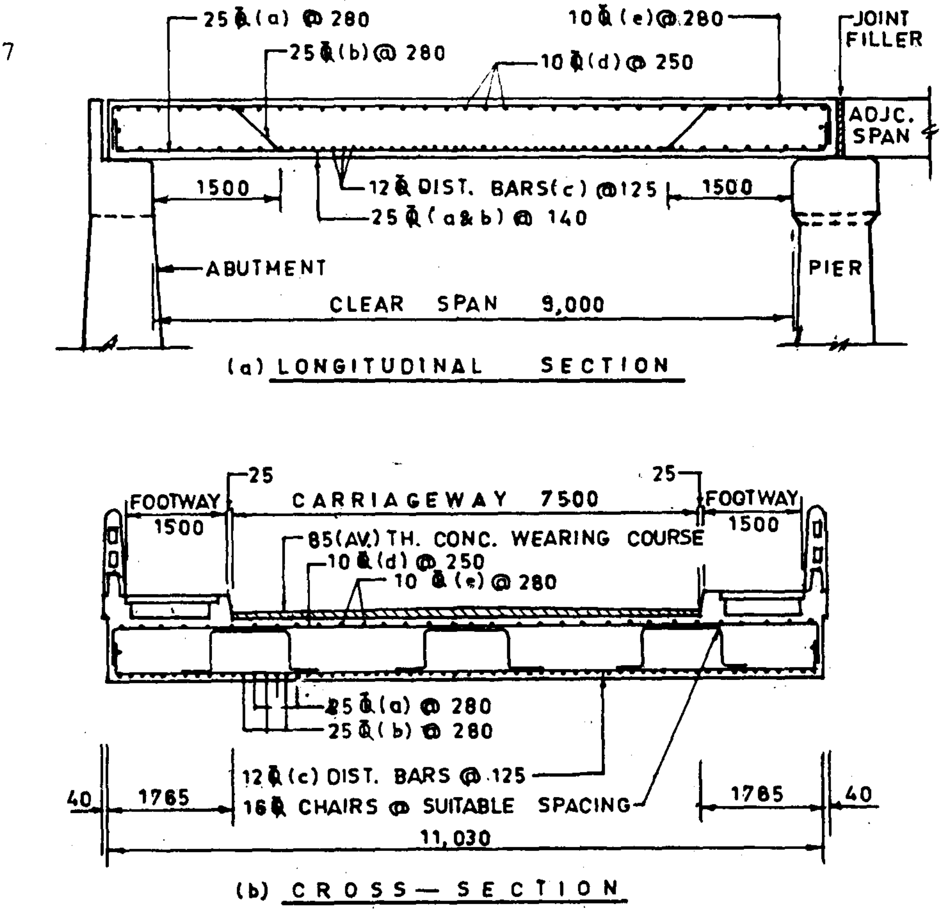

FIG. 7·6 DETAILS OF REINFORCEMENT FOR A 9·0 m CLEAR SPAN SOLID SLAB BRIDGE

Shear due to footway loading :

Shear = $\frac{1}{2} \times 9.63 \times 106 = 509$ Kg/metre width

Design shear = D.L. Shear + L.L. Shear + Footway Shear = 8873 + 6050 + 509 = 15,432 Kg.
= 15,432 × 9.8 = 1,51,200 N

As per clause 304.7.1 of IRC bridge code, Section III (IRC : 21-1987), Shear stress = $\frac{V}{bd}$

Shear stress = $\frac{1,51,200}{1000 \times 632} = 0.24$ MP$_a$

Basic permissible shear stress as per clause 304.7.3 of IRC:21-1987 for M20 concrete is 0.34 MP$_a$. Hence no shear reinforcement is necessary.

Check for bond failure :

To prevent bond failure, adequate anchorage length shall be provided for all the tensile reinforcement at the ends as recommended in IRC:21-1987.

7.3 Details of some slab bridges having various span lengths are reproduced from the ***"Standard Plans for Highway Bridges –Vol. II – Concrete Slab Bridges"*** published by the Ministry of Shipping and Transport (Roads Wing). Grade of concrete is M20 and reinforcing steel is HYSD bars as in the illustrative example 7.1. For further details, the standard plans may be referred to.

TABLE 7.1 : DETAILS OF SOME SLAB BRIDGES HAVING VARIOUS SPAN LENGTHS WITH 1.5 METRES FOOT-PATHS *(MOS&T)* [2]

Sl. No.	Effective span (m)	Overall depth of slab (mm.)	Main reinforcement		Distribution reinforcement		Top steel – both longitudinal and transverse (HYSD bars)
			Dia. (mm)	Spacing (mm)	Dia (mm)	Spacing (mm)	
1	3.37	315	16	125	10	115	10 Φ @ 250 mm and 10 Φ @ 300 mm
2	4.37	365	16	105	10	100	10 Φ @ 210 mm and 10 Φ @ 300 mm
3	5.37	410	20	145	10	90	10 Φ @ 290 mm and 10 Φ @ 300 mm
4	6.37	470	20	125	10	90	10 Φ @ 250 mm and 10 Φ @ 300 mm
5	7.37	530	20	110	10	95	10 Φ @ 220 mm and 10 Φ @ 300 mm
6	8.37	595	20	95	10	95	10 Φ @ 190 mm and 10 Φ @ 300 mm
7	9.37	665	25	135	10	95	10 Φ @ 405 mm and 10 Φ @ 300 mm
8	10.37	740	25	120	10	95	10 Φ @ 360 mm and 10 Φ @ 300 mm

7.4 REFERENCES

1. IRC:21-1987 : Standard specifications and Code of Practice for Road Bridges, Section III – Cement Concrete (Plain and Reinforced) (Second Edition) – Indian Roads Congress.
2. Ministry of Shipping and Transport (Road Wing) : Standard Plans for Highway Bridges, Volume II, Concrete Slab Bridges, (First Revision), 1983 – Indian Roads Congress.
3. Dayaratnam, P. – *"Design of Reinforced Concrete Structures (Second Edition)"* – Oxford & IBH Publishing Co.
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Chapter 8

SLAB AND GIRDER BRIDGES

8.1 GENERAL

8.1.1 Slab and girder bridges are used when the economical span limit of solid slab bridges is exceeded. For simply supported spans, this limit is generally found to be nearly 10 metres and for continuous or balanced cantilever type structures, this limit is 20 to 25 metres.

8.1.2 The deck slab of a slab and girder bridge spans transversely over the girders which run longitudinally spanning between abutment or pier supports. The spacing of the girders depends on the number of girders to be provided in the deck which again is related to the cost of materials, shuttering, staging etc. Closer beam spacing means lesser thickness of deck slab and consequently savings in concrete and steel in deck slab but since the number of beams is more in that case, this increases the quantity of concrete, shuttering and reinforcement for girders and for longer spans where bearings are required, the number of bearings. Therefore, the most economical arrangement of bridge deck varies from place to place depending on the cost of materials, shuttering, staging etc., in that locality. It has been observed that three beams deck is generally found economical than two, four or five beams deck having a carriageway for two lanes. The girder spacings in such cases are usually between 2.25 to 2.75 metres.

8.1.3 The cross-beams or diapharagms are used in a bridge deck for the following reasons :

i) To distribute loads between main girders.

ii) To offer resistance to torsion of main girders.

iii) To stiffen the girders laterally.

For proper functioning, at least two cross-beams at two ends and one at the centre are essential. A spacing of about 4.5 m. to 6.0 m. is generally found satisfactory.

8.1.4 Sometimes in long bridges, it is necessary to keep provision for carrying pipes (gas, oil or water), cables etc., through the bridge deck for which space under the footway may be utilised as shown in Fig. 8.1.

8.2 DESIGN OF DECK SLAB

8.2.1 If no gap between the deck slab and the cross beams is maintained, the slab panel becomes a two-way slab continuous in both the direction. In two-way slab, the live load moments due to a concentrated or locally distributed load may be worked out by *"Pigeaud's Method"* but when the deck slab is not made monolithic with the cross beam i.e. when a gap is maintained between the deck slab and the cross-beam, the slab may be designed as one way slab as illustrated in example 7.1 in Chapter 7. Since the dead load of the deck is uniformly distributed over the whole area, the method outlined by *"Rankine & Grashoff"* may be adopted in finding out the dead load moments.

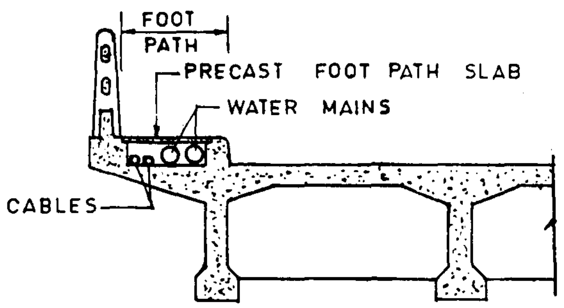


FIG. 8·1 ARRANGEMENT FOR WATER MAINS, ELECTRIC CABLES ETC

8.3 DESIGN OF GIRDERS

8.3.1 In designing the girders, the dead load of deck slab, cross-beams, wearing course, wheel guard, railings etc., may be equally distributed over the girders. The distribution of the live loads, on the otherhand, is not a simple one. It depends on many factors such as the span-width ratio, properties of the bridge deck and the position of the live loads on the girders. Therefore, the sharing or distribution of live loads on the girders and consequently the live load moment varies from girder to girder and as such this aspect requires to be considered carefully. This subject has already been dealt with in Chapter 6.

ILLUSTRATIVE EXAMPLE 8.1

Design a slab and girder bridge with 7.5 m. clear roadway having a span of 12.0 m. between centre line of bearings. The deck may consist of 3 girders spaced at 2.45 m. centres. The bridge deck will have no footpaths. Loading – Single lane of Class 70-R or two lanes of Class A.

Let the cross-section of the deck be assumed as shown in Fig. 8.2a.

Design of Deck Slab

Since the deck slab is monolithic with the cross-beams, it will be designed as a two-way slab supported on longitudinal girders and cross-beams with continuity on all sides. As per Clause 305.1.3 of IRC Bridge Code Section III (IRC:21-1987), clear span may be adopted in the design.

Dead Load Moments

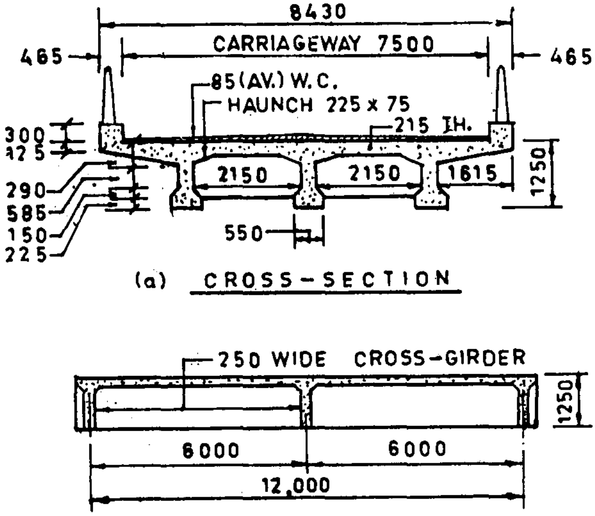
Dead Load

i) Due to self weight of 225 mm (average) thick slab @ 2400 kg./m^3		540 Kg./m^2
ii) 85 mm. (average) wearing course @ 2500 Kg./m^3		213 Kg./m^2
		753 Kg./m^2
	Say	775 Kg./m^2

Using Rankine-Grashoff formula,

Load in the shorter direction, $w_2 = \dfrac{w\,r^4}{1+r^4}$, Where w = Total load per sq.m.

$$r = \frac{L}{B} = \frac{\text{Longer Span}}{\text{Shorter Span}} = \frac{5750}{2150} = 2.67$$



(a) CROSS-SECTION

(b) LONGITUDINAL SECTION

FIG. 8·2 DETAILS OF SLAB AND GIRDER BRIDGE (Example 8·1)

$$\therefore\ w_2 = 775 \times \frac{(2.67)^4}{1+(2.67)^4} = 775 \times \frac{50.82}{51.82} = 760 \text{ Kg./m}^2$$

And load in the longer direction = $(w - w_2) = (775 - 760) = 15$ Kg./m^2.

Maximum positive moment at mid span in the shorter (transverse) direction

$$= \frac{w_2 B^2}{16} = \frac{760 \times (2.15)^2}{16} = 220 \text{ Kgm. per metre width}$$

Maximum positive moment at midspan in the longer (longitudinal) direction,

$$= \frac{w_1 L^2}{16} = \frac{15 \times (5.75)^2}{16} = 31 \text{ Kgm.}$$

Maximum negative moment at support in the shorter direction,

$$= \frac{w_2 B^2}{8} = \frac{760 \times (2.15)^2}{8} = 439 \text{ Kgm.}$$

Maximum negative moment at support in the longer direction,

$$= \frac{w_1 L^2}{8} = \frac{15 \times (5.75)^2}{8} = 62 \text{ Kgm.}$$

Live Load Moments

Since it is a two-way slab, the live load moments will be determined by using Pigeaud's method with Poisson's ratio of concrete to be 0.15 as advocated in the IRC Bridge Code, Section III (IRC:21-1987) Clause 305.13.1.

Pigeaud's Method

The method outlined by M. Pigeaud deals with the effect of concentrated load on slabs spanning in two directions or on slab spanning in one direction where the width-span ratio exceeds 3. A synopsis of the method is given here.

The dispersion of the load may be found out as per following equations :

$$U = \sqrt{(a + 2d)^2 + H^2} \quad (8.1)$$

$$V = \sqrt{(b + 2d)^2 + H^2} \quad (8.2)$$

Where U = Dispersion of load along shorter span
V = Dispersion of load along longer span
a = Dimension of tyre contact along shorter span
b = Dimension of tyre contact along longer span
d = Thickness of wearing coarse
H = Thickness of deck slab

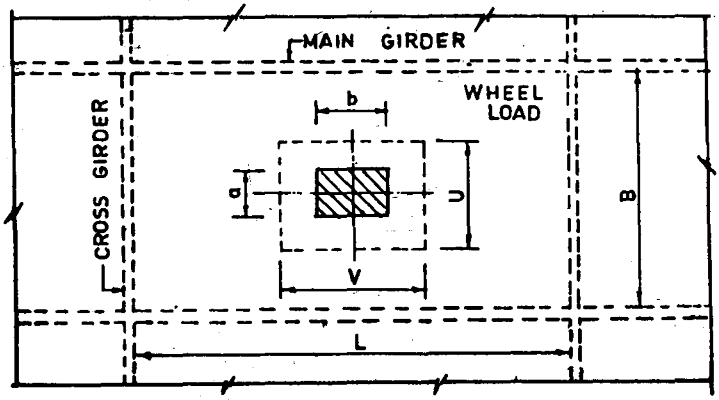


FIG. 8·3 DISPERSION OF LIVE LOAD ON DECK SLAB

Having got the values of U and V, the ratio of $\frac{U}{B}$ and $\frac{V}{L}$ may be determined. The values of the coefficients m_1 and m_2 are obtained from the curves (Fig. C-1 to C-8 in Appendix C) when the values of $\frac{U}{B}, \frac{V}{L}$ and K $(= \frac{B}{L} = \frac{\text{Shorter Span}}{\text{Longer Span}})$ are known.

Moment in the shorter (transverse) direction per metre width = $W (m_1 + \mu m_2) = W (m_1 + 0.15\, m_2)$ Kgm. and moment in the longer (longitudinal) direction per metre width = $W (m_2 + \mu m_1) = W (m_2 + 0.15\, m_1)$ Kgm. where W is the total load. It has been advocated that due to continuity, the mid-span moments may be reduced by 20 percent and the same moment may be taken as the support (negative) moment also. In the example, Class 70-R tracked vehicle will govern the design.

$U = \sqrt{(0.84 + 2 \times 0.085)^2 + (0.215)^2} = 1.03$ m.

$V = \sqrt{(4.57 + 2 \times 0.085)^2 + (0.215)^2} = 4.73$ m.

$K = \frac{B}{L} = \frac{2.15}{5.75} = 0.37$ say 0.4 ; $\frac{U}{B} = \frac{1.03}{2.15} = 0.48$; $\frac{V}{L} = \frac{4.73}{5.75} = 0.82$

From curve C-1 (Appendix C), $m_1 = 0.081$, $m_2 = 0.007$

Moment in the transverse direction per metre width = W (m_1 + 0.15 m_2).

= 35 × 1000 (0.081 + 0.15 × 0.007) = 2872 Kgm.

Moment in the longitudinal direction per metre width = W (m_2 + 0.15 m_1)

= 35 × 1000 (0.007 + 0.15 × 0.081) = 670 Kgm.

Taking mid span and support moment as 80 percent of the above as stated before and allowing for 25 percent impact as per Art. 5.6,

Span and support moment in the transverse direction per metre = 2872 × 0.8 × 1.25 = 2872 Kgm.

Span and support moment in the longitudinal direction per metre = 670 × 0.8 × 1.25 = 670 Kgm.

Design Moments per Metre

a) ***Transverse direction***

i) At mid span, design moment = D.L.M. + L.L.M. = 220 + 2872 = 3092 Kgm. = 30,300 Nm.

ii) At support, design moment = -439 - 2872 = -3311 Kgm. = -32,450 Nm.

b) ***Longitudinal direction***

i) At mid span, design moment = 31 + 670 = 701 Kgm. = 6900 Nm.

ii) At support design moment = -62 -670 = -732 Kgm. = -7200 Nm.

Depth of Slab & Reinforcement

a) At mid-span, $d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{30{,}300 \times 10^3}{0.95 \times 10^3}} = 178$ mm.

Overall depth assumed = 215 mm.

∴ Effective depth = 215 – 30 (cover) – 6 (½ dia of bar) = 215 – 36 = 179 mm.

Hence the depth is satisfactory. HYSD bars are used both in the transverse & longitudinal direction.

∴ Area of steel in the transverse direction $= \frac{30{,}300 \times 10^3}{200 \times 0.894 \times 179} = 947 \text{ mm}^2$

Provide 12 Φ @ 110 mm. (As = 1027 mm ^{2})

Effective depth in the longitudinal direction = 179 – 12 = 167 mm.

∴ Area of steel in the longitudinal direction $= \frac{6900 \times 10^3}{200 \times 0.894 \times 167} = 231 \text{ mm}^2$

Provide 8 Φ @ 100 mm. (As = 500 mm ^{2})

b) Overall depth at support (considering portion of haunch below 1:3 line as per Clause 305.2.2 of IRC:21-1987) = $\left(231 + \frac{225}{3}\right)$ = 306 mm. Therefore, effective depth = 306 – 36 = 270 mm.

Area of steel in the transverse direction = $\frac{32{,}450 \times 10^3}{200 \times 0.894 \times 270} = 661\,\text{mm}^2$; Provide 12 Φ @ 110 mm (As = 1027 mm^2)

Distribution steel as per Clause 305.15 of IRC:21-1987 shall be to cater for a moment equal to (0.3 LLM + 0.2 other moments). Let us provide 30 per cent of main steel i.e. 0.3 × 661 = 198 mm^2. Provide 8 Φ @ 100 (As = 500 mm^2).

Effective depth in the longitudinal direction = 215 – 30 – 12 – 4 = 169 mm

∴ Area of steel in the longitudinal direction = $\frac{7200 \times 10^3}{200 \times 0.894 \times 169} = 238\,\text{mm}^2$

Provide 8 Φ @ 100 mm. (As = 500 mm^2)

Design of Cantilever

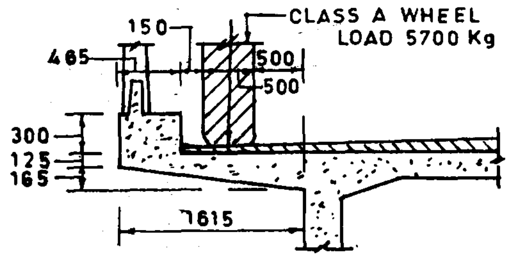


FIG. 8·4 WHEEL LOAD ON CANTILEVER DECK SLAB (Example 8·1)

Dead Load Moment at face of girder

i) Due to railing = 150 Kg × 1.507 m.	226 Kgm.
ii) Due to wheel guard etc. = 0.465 × 0.3 × 2400 × 1.383	463 Kgm.
iii) Due to slab = 0.125 × 1.615 × 2400 × 0.807	391 Kgm.
$\frac{1}{2}$ × 0.165 × 1.615 × 2400 × 0.538	172 Kgm.
	1252 Kgm.

Live Load Moment at face of girder

The effect of Class 70-R tracked or wheel load will not be maximum since it is to be placed 1.2 m. away from the wheel guard. Class A wheel load as shown in Fig. 8.4 will produce worst effect and will therefore, govern the design.

Dispersion of wheel load parallel to the supporting girders (vide Chapter 5),

Effective width, $b_e = 1.2 \times W$

$$= 1.2 \times 0.75 + [0.25\ (av) + 2 \times 0.085] = (0.90 + 0.42) = 1.32\ m.$$

$\therefore$ Live load moment at face of girder per metre width $= \dfrac{5700 \times 0.75}{1.32} = 3239$ Kgm.

Impact percentage from Fig. 5.3 = 50%. $\therefore$ LLM with impact = $1.5 \times 3229 = 4859$ Kgm.

$\therefore$ Design moment at face of girder = DLM + LLM = 1252 + 4859 = 6111 Kgm. = 59,900 Nm.

$\therefore d = \sqrt{\dfrac{59{,}900 \times 10^3}{0.95 \times 10^3}} = 251$ mm. ; Overall depth provided = 290 mm.

$\therefore$ Effective depth = 290 – 30 – 8 = 252 mm. Hence the depth is satisfactory.

Area of Steel $= \dfrac{59.900 \times 10^3}{200 \times 0.894 \times 251} = 1335\ mm^2$. Provide 12 Φ @ 220 mm and 16 Φ @ 220 mm in between ($As = 1427\ mm^2$)

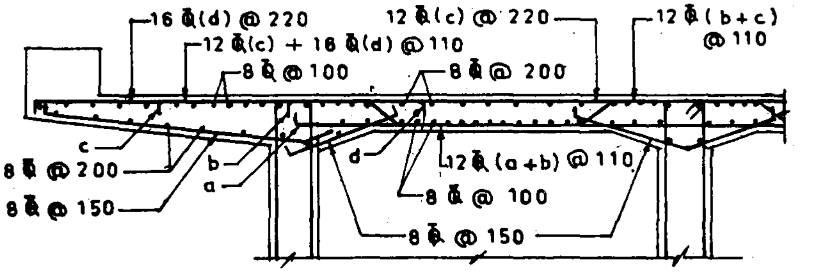


FIG. 8·5 DETAILS OF REINFORCEMENT IN DECK SLAB (Example 8·1)

Design of Girders

a) *Dead Load Moments*

Load per metre of deck

i)	Deck slab between girders = 5.2 × 0.215 × 1.0 × 2400	2683 Kg.
ii)	Cantilever slab = 2 × ½ (0.125 + 0.290) × 1.615 × 2400	1609 Kg.
iii)	Haunches = 4 × ½ × 0.225 × 0.075 × 2400	81 Kg.
iv)	Wheel guard etc. = 2 × 0.3 × 0.465 × 2400	670 Kg.
v)	T-beam web = 3 × 0.660 × 0.3 × 2400	1426 Kg.
	Bottom bulb = 3 × ½ ×(0.3 + 0.55) × 0.15 × 2400 + 3 × 0.55 × 0.225 × 2400	1350 Kg.
vi)	Wearing course = 7.5 × 0.085 × 2500	1594 Kg.
vii)	Railing = 2 × 150	300 Kg.
		9713 Kg.
	Say	9720 Kg.

Effective span = 12.0 m.

$\therefore$ Maximum dead load moment at mid span due to uniformly distributed load

$$= 9720 \times \frac{(12.0)^2}{8} = 1{,}74{,}960 \text{ Kgm.}$$

Weight of diaphragm or cross-beam
$= 2 \times 2.15 \times 0.810 \times 0.25 \times 2400 = 2090$ Kg.

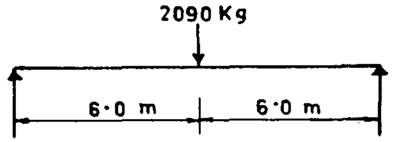


FIG. 8·6 (Example 8·1)

Dead load moment at mid span due to cross-beam at centre $= \dfrac{2090 \times 12.0}{4} = 6270$ Kgm.

Total DLM = 1,74,960 + 6270 = 1,81,230 Kgm.

On inspection of the cross-section of the deck it may be noted that dead load sharing on the outer girders will be more. Let us assume that outer girders take $^3/_8$th each and central girder $^1/_4$ th of the total load.

$\therefore$ DLM on outer girder $= \frac{3}{8} \times 1{,}81{,}230 = 67{,}960$ Kgm.

DLM on central girder $= \frac{1}{4} \times 1{,}81{,}230 = 45{,}300$ Kgm.

Live Load Moments

a) ***Single Lane of Class 70-R (tracked) Loading***

Live load moment at midspan

= Area of the influence line diagram × intensity of load

$= 2 \times \frac{1}{2}(3.0 + 1.8575) \times 2.285 \times 15{,}317$

= 1,70,000 Kgm.

Impact factor = 10 percent $\therefore$ LLM with impact = $1.1 \times 1{,}70{,}000 = 1{,}87{,}000$ Kgm.

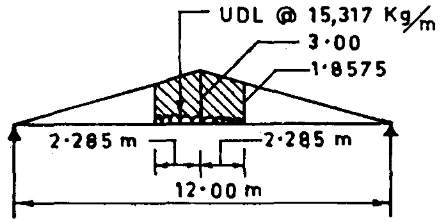


FIG. 8·7 CLASS 70 R (TRACKED) LOADING ON INF. LINE DIAG. FOR MOMENT AT MID-SPAN (Example 8·1)

b) ***Two Lanes of Class A Loading***

Live load moment at midspan for single lane of IRC Class A Loading

$$= \frac{3.0}{6.0}[1.7 \times 6.8 + (6.0 + 4.8) \times 11.4 + (1.1 + 0.5) \times 2.7]$$

$= \frac{1}{2}(11.56 \times 123.12 + 4.32) = 69.5$ tm = 69,500 Kgm.

For two lanes of Class A loading, LLM = 2 × 69,500 = 1,39,000 Kgm.

Impact factor from equation 5.5 $= \dfrac{4.5}{6 + L} = \dfrac{4.5}{6 + 12} = 0.25$

$\therefore$ LL Moment with impact = $1.25 \times 1{,}39{,}000 = 1{,}73{,}750$ Kgm.

This moment is less than that due to single lane of Class 70-R loading. Hence the former governs the design.

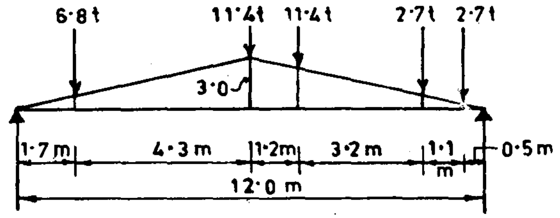


FIG. 8·8 CLASS A LOADING ON INF. LINE DIAG. FOR MOMENT AT MID SPAN (Example 8·1)

Distribution of Live Load Moments on the Girders

As already explained in Chapter 6, the live load and consequently the live load moment will be distributed over the girders in varying proportions depending on the properties of the deck. Since in this case the span-width ratio is less than 2, Morice and Little's simplified method of load distribution will be used.

For this particular bridge deck, the distribution co-efficients have already been worked out by the Morice & Little's simplified method for single lane of IRC Class AA (tracked) loading in Chapter 6. The same values may be taken for Class 70-R (tracked) loading also.

Live load moment on outer girder $= \frac{1{,}87{,}000}{3} \times 1.45 = 90{,}380$ Kgm.

Live load moment on central girder $= \frac{1{,}87{,}000}{3} \times 1.11 = 69{,}190$ Kgm.

$\therefore$ Total design moment for outer girder = DLM + LLM = 67,960 + 90,380 = 1,58,340 Kgm. = 15,51,700 Nm.

Total design moment for central girder = DLM + LLM = 45,300 + 69,190 = 1,14,490 Kgm. = 11,22,000 Nm.

Design of T-beam

a) ***Outer girder***

The outer girder has an overhang of 1.765 m. from the centre line of girder and centre to centre distance of girders is 2.45 m. Therefore, the outer girder is also a T-beam. The average thickness of the overhang is 235 mm. in place of the slab thickness of 215 mm. on the inner side. Therefore, the effective width of flange for T-beam in terms of Clause 305.12.2 of IRC : 21-1987 is valid for the outer girder.

The effective flange width shall be the least of the following :

i) $^1/_4$ of span $= \frac{1}{4} \times 12.0 = 3.00$ m.

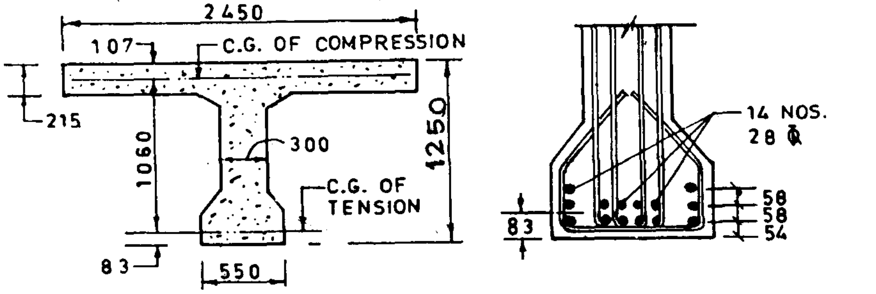
ii) Centre to centre distance of beam, i.e. 2.45 m.

iii) Breadth of web plus 12 times slab thickness $= 0.3 + 12 \times 0.215 = 2.88$ m.

Hence 2.45 m. shall be the effective flange width. The section of the outer girder is shown in Fig. 8.9.

$\sigma_c = 6.7$ MPa ; Average σ_c in the flange may be taken as $0.8 \times 6.7 = 5.36$ MPa

$\sigma_s = 200$ MPa. Average steel stress will be $200 \times \frac{1060}{1088} = 196$ MPa



(a) EFFECTIVE SECTION OF MAIN GIRDER (b) ARRANGEMENT OF TENSILE REINFORCEMENT

FIG. 8·9 (Example 8·1)

∴ Moment of resistance of compression = $2450 \times 215 \times 5.36 \times 1060$ Nmm
= $29{,}93{,}000 \times 10^3$ Nmm. = 29,93,000 Nm.

This is more than the design moment of 15,51,700 Nm.

$As = \dfrac{15{,}51{,}700 \times 10^3}{196 \times 1060} = 7469\ mm^2$ Provide 14 nos. 28 Φ HYSD bars ($As = 8610\ mm^2$)

Minimum tension reinforcement as per Clause 305.16 of IRC : 21-1987 = $\dfrac{0.2}{100} \times 300 \times 1167 = 700\ mm^2$

b) *Central Girder*

The section of the girder is the same as that of the outer girder but the design moment is less. Hence, the section is safe in compression. Reinforcement for central girder, $As = \dfrac{11{,}22{,}000 \times 10^3}{196 \times 1060} = 5400\ mm^2$

Provide 12 Nos. 28 Φ HYSD bars ($As = 7380\ mm^2$)

Shear and Shear Reinforcement near support

a) *Dead Load Shear*

Total UDL per metre of bridge = 9720 Kg.

Shear taken by outer girder = $\frac{3}{8} \times 9720 \times 6.0 = 21{,}870$ Kg.

Shear taken by central girder = $\frac{1}{4} \times 9720 \times 6.0 = 14{,}580$ Kg.

Dead load shear due to weight of cross beam on outer girder = $^1/_4$ of total shear = $\frac{1}{4} \times \frac{1}{2} \times 2090 = 260$ Kg.

D.L. shear due to cross-beam on central girder = $\frac{1}{2} \times \frac{1}{2} \times 2090 = 520$ Kg.

∴ Total D.L. shear on outer girder = 21,870 + 260 = 22,130 Kg.

Total D.L. shear on central girder = 14,580 + 520 = 15,000 Kg.

b) *Live Load Shaer*

Shear for live load within 5.5 m. of either supports will be maximum. For outer girder, shear will be calculated as per clause 305.9.2(i) (b) and shear for the central girder will be evaluated as per Clause 305.9.2(i) (a) of IRC : 21-1987.

c) *Live Load Shear on Outer Girder*

Since the distribution coefficient will be more for the outer girder when load is placed near the centre, Class 70-R loading is placed at a distance of 6.0 m i.e. at the centre of the span. Therefore, reaction of each support and as such the total L.L. shear will be 35.0 tonnes = 35,000 Kg.

L.L. shear on the outer girder = Distribution coefficient × average L.L. shear = $1.45 \times \frac{35,000}{3} = 16,916$ Kg..

With 10 percent impact, L.L. shear on outer girder = $1.1 \times 16,916 = 18,600$ Kg.

d) *Design Shear for Outer Girder*

Design Shear = D.L. Shear + L.L. Shear = 22,130 + 18,600 = 40,700 Kg. = 3,99,200 N.

Shear stress = $\frac{V}{bd} = \frac{3,99,200}{300 \times 1060} = 1.26$ MP.

As per Clause 304.7 of IRC : 21-1987, permissible shear stresses for M20 concrete

i) Without shear reinforcement = 0.34 MP.

ii) With shear reinforcement = $0.07 \times 20 = 1.40$ MP.

Hence, the section will be safe with shear reinforcement.

Shear Reinforcement for Outer Girder

Bent up bars

Shear resistance of 2 - 28 Φ bent up bars in double system $= 2 \times 2 \times 615 \times 200 \times 0.707 = 3,47,800$ N

However, as per Clause 304.7.4.1(ii) of IRC : 21-1987, not more than 50 percent of the shear shall be carried by bent up bars. Hence shear to be carried by bent up bars = $\frac{1}{2} \times 3,99,200 = 1,99,600$ *N* and shear to be carried by stirrups = 1,99,600 N

Using 10 Φ 4 legged stirrups, @ 175 mm

$$Asw = \frac{V \cdot s}{\sigma_s \cdot d} = \frac{1,99,600 \times 175}{200 \times 1060} = 166 \text{ mm}^2$$

Asw provided = $4 \times 78 = 312$ mm^2

Shear Reinforcement for Other Sections

The shears at various sections shall be calculated and shear reinforcement shall be provided accordingly as explained above.

e) *Live Load Shear for Central Girder*

Class 70-R tracked loading when placed near the support will produce maximum effect (Fig. 8.10).

$$R_A = \frac{70,000 \times 9.715}{12.0} = 56,670 \text{ Kg.}$$

∴ Shear at A = R_A = 56,670 Kg.

Shear with 10 percent impact = $1.1 \times 56,670 = 62,340$ Kgs.

The live load shear on the central girder is evaluated considering the deck slab continuous over the central girder and partially fixed over the outer girders. In such case, the sharing of the shear may be assumed as 0.25 on each outer girders and 0.5 on the central girder.

Hence, L.L. shear on the central girder $= 0.5 \times 62{,}337 = 31{,}170$ Kg.

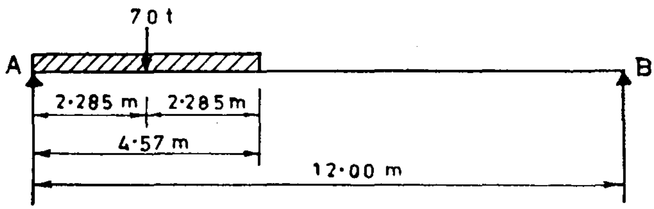


FIG. 8·10 LOADING FOR MAX. LIVE LOAD SHEAR (Example 8·1)

f) ***Design Shear for Central Girder***

Design Shear = DL Shear + LL Shear = 15,000 + 31,170 = 46,270 Kg. = 4,53,500 N.

$\therefore$ Shear stress $= \dfrac{V}{bd} = \dfrac{4{,}53{,}500}{300 \times 1060} = 1.43$ MP$_a$

This exceeds the permissible limit of shear stress of 1.40 MP$_a$ with shear reinforcement. Hence the section is to be modified.

Let us widen the web section near the support to same as the bottom bulb as shown in Fig. 8.11.

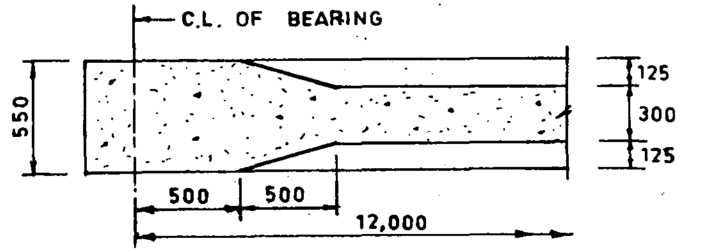


FIG. 8·11 WIDENING OF WEB NEAR SUPPORT (Ex. 8·1)

Extra DL shear due to the widening of the web as in Fig. 8.11
$= (0.5 \times 0.735 \times 0.25 + \frac{1}{2} \times 0.5 \times 0.735 \times 0.25) \times 2400 = 220 + 110 = 330$Kg. = 3200 N.

$\therefore$ Modified design shear = 4,53,500 + 3200 = 4,56,700 N.

Revised shear stress $= \dfrac{4{,}56{,}700}{500 \times 1060} = 0.78$ MP$_a$

Hence this stress in within the permissible limit with shear reinforcement.

Shear Reinforcement for Central Girder

Bent up bars

Shear resistance of 2 Nos. 28 Φ bent up bars in double system as in outer girder = 3,47,800 N. However, not more than 50 percent of the design shear shall be carried by the bent up bars. Hence, shear to be resisted by bent up bars and stirrups is $\frac{1}{2} \times 4{,}56{,}700 = 2{,}28{,}350$ N. each. With a stirrup spacing of 175 mm,

$$\therefore \text{Asw} = \frac{V \cdot s}{\sigma_s \cdot d} = \frac{2{,}28{,}350 \times 175}{200 \times 1060} = 188 \text{ mm}^2$$

If 10 Φ 4 legged stirrups are used, Asw provided = 4 × 78 = 312 mm ^{2}

Shear at a distance of 2.5 m. (i.e. where normal width of web of 300 is available and where the shear resistance of bent up bars is not effective).

D.L. shear at support = 15,100 Kg.

Less load on 2.5 m length i.e. $\frac{1}{4} \times 9700 \times 2.5 = 6075$ Kg.

D.L. shear at the section = 15,100 – 6075 = 9025 Kg.

L.L. shear at 2.5 m from support

$$R_A = \frac{70{,}000 \times 7.215}{12.0} = 42{,}100 \text{ Kg.}$$

L.L. shear at the section with impact
= 1.1 × 42,100 = 46,300 Kg.

L.L. shear on the central girder
= $\frac{1}{2} \times 46{,}300 = 23{,}150$ Kg.

∴ Design shear = D.L. shear + L.L. shear
= 9025 + 23,150 = 32,175 Kg. = 3,15,300 N.

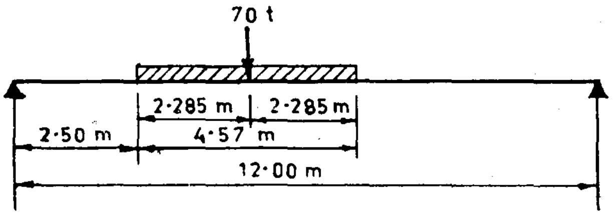


FIG. 8·12

$$\therefore \text{Shear stress} = \frac{V}{bd} = \frac{3{,}15{,}300}{3006 \times 1060} = 0.99 \text{ MP}_a$$

This is within permissible limit of 1.40 MP$_a$ with shear reinforcement.

Using 10 Φ 4 legged stirrups @ 175 mm

$$\text{Asw} = \frac{V \cdot S}{\sigma_S \cdot d} = \frac{3{,}15{,}300 \times 175}{200 \times 1060} = 260 \text{ mm}^2$$; Asw provided = 312 mm ^{2}. Hence, satisfactory.

Shear reinforcement at other sections of the girder shall be worked out on the same principles as outlined above.

Minimum Side-face Reinforcement

As per Clause 25.5.1.3 of IS : 456-1978, minimum side-face reinforcement on both the faces shall be equal to 0.1 percent of the web area.

$$\therefore \text{Reinforcement per metre depth} = \frac{0.1}{100} \times 300 \times 1000 = 300 \text{ mm}^2$$

Provide 6 dia. m.s. bars @ 150 mm (As = 375 mm ^{2}).

Reinforcement details of central girder are shown in Fig. 8.13.

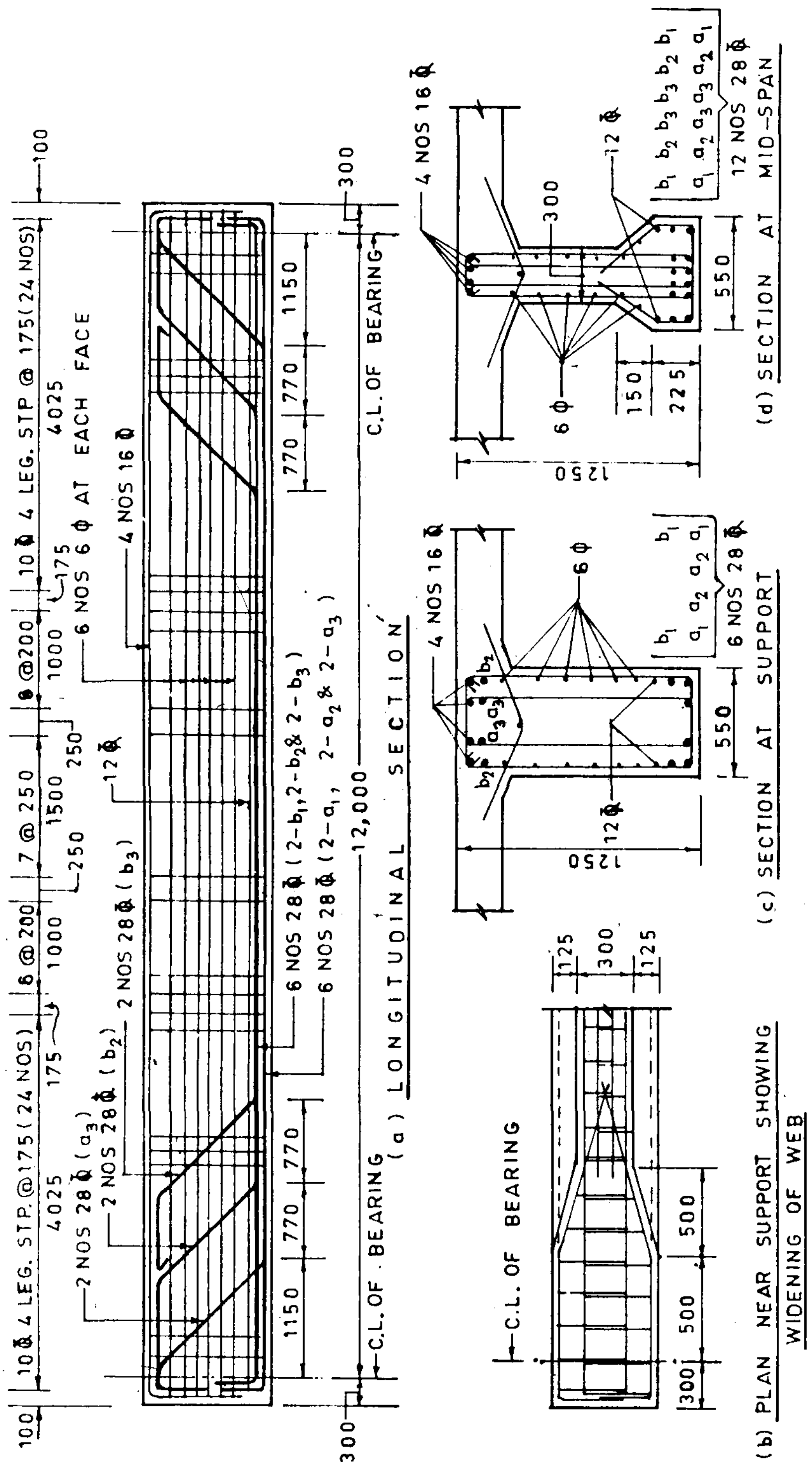


FIG. 8·13 DETAILS OF REINFORCEMENT FOR CENTRAL GIRDER (Example 8·1)

Design of Cross-beams

Since the span-width ratio of the deck is less than 2, the transverse deck is not rigid and therefore the central cross-beam is designed by Morice and Little's simplified Method.

Dead Load Moments

As already explained in Chapter 6, maximum transverse moment per metre length of the deck at centre is given by :

$$M_y = b\,[\mu_0 r_1 - \mu_{30} r_3 + \mu_{50} r_5] \tag{8.3}$$

Where r_n (= 1,3,5) $= \dfrac{4w}{n\pi} \sin \dfrac{n\pi u}{2a} \sin \dfrac{n\pi c}{2a}$

Now the transverse deck is subjected to moments due to the following dead loads –

a) Udl due to wt. of deck slab and wearing course spread throughout the length and the breadth of the deck.

b) Udl due to weight of main beams acting along longitudinal direction but point load along transverse direction.

c) Udl due to self wt. of cross beam acting along transverse direction but point load along longitudinal direction.

a) ***Udl due to deck slab and wearing course***

To find out the transverse moment due to load of item (a) above, the equivalent deck of width 7.35 m. (vide Chapter 6) may be divided into a number of equal parts, say 4 equal parts each of 1.84 m. width and the effect of each load on the transverse deck acting at the c.g. of each part may be summed up and the transverse moment may be obtained from equation 8.3 assuming u = c = a.

The full details of the calculations are omitted here but important steps are shown. It may be noted here that the procedure of finding out the μ values is the same as already outlined in Chapter 6 The influence line curves shown in Fig. 6.10 will also be applicable here since the same deck has been analysed there.

Load per metre of deck excepting wt. of T-beam as worked out before = 6944 Kg.

Dividing the equivalent width into 4 equal parts, load per part $= \dfrac{6944}{4} = 1736$ Kg.

Σ μ values from Fig. 6.10 at c.g. of each load are given below :

$\mu_0 = 0.020$, $\mu_{30} = (-)0.02$ and $\mu_{50} = 0$

Since u = c = a, $\sin \dfrac{n\pi u}{2a} \sin \dfrac{n\pi c}{2a} = 1$ when n = 1, 3, or 5

$$\therefore\ M_y = b\,[\mu_0 r_1 - \mu_{30} r_3 + \mu_{50} r_5] = \frac{4\,bw}{\pi}\left[\mu_0 - \frac{\mu_{30}}{3} + \frac{\mu_{50}}{5}\right]$$

$$= \frac{4 \times 3.675 \times 1736}{\pi}\left[(0.02) - \frac{(-0.02)}{3} + \frac{(0)}{5}\right] = 4069\,(0.02 + 0.007) = 220 \text{ Kgm.}$$

∴ Moment on each cross beam = 220 × spacing of cross beam = 220 × 6.0 = 1320 Kgm.

b) *Udl due to wt. of main beam*

In this case, the Udl is distributed throughout the length but the wt. of the beams acts on the transverse deck at beam positions. The transverse moment coefficients may be obtained from the influence line curves (Fig. 6.10) corresponding to the beam positions, weight of each beam per metre run is equal to 925 Kg. as calculated before.

$\Sigma\,\mu$ values from Fig. 6.10 at beam position are as below :

$\mu_\theta = 0.11$, $\mu_{3\theta} = 0.04$ and $\mu_{5\theta} = 0.02$

The values of $\sin\frac{n\pi u}{2a}\sin\frac{n\pi c}{2a}$ are the same as item (a) above i.e. equal to unity.

$$\therefore\ M_y = \frac{4\times 13{,}675\times 925}{\pi}\left[0.11-\frac{0.04}{3}+\frac{0.02}{5}\right] = 4330\times 0.101 = 437\ \text{Kgm.}$$

Moment on cross beam = 437 × 6.0 = 2662 Kgm.

c) *Self wt. of cross beam*

The cross beams may be divided into 4 equal parts the wt. of each part is assumed to act at its centre of gravity.

Wt. of each part = $\frac{1}{4}$ (2090) = 520 Kg.

$\Sigma\,\mu$ values from Fig. 6.10 at c.g. of each load are :

$\mu_\theta = 0.020$, $\mu_{3\theta} = (-)0.02$ and $\mu_{5\theta} = 0$ as before.

Value of loading series from Fig. B-13 (Appendix B)

Cross beam no.	u	$\frac{u}{2a}$	Values of $\sin\frac{n\pi u}{2a}$ from Fig. B-13		
			n = 1	n = 3	n = 5
	6.0	0.5	1.0	(-)1.0	1.0

$M_y = b\,[\mu_\theta r_1 - \mu_{3\theta} r_3 + \mu_{5\theta} r_5]$

Putting values of r from equation 6.6,

$$M_y = \frac{Wb}{a}\left[\mu_\theta \sin\frac{\pi u}{2a} - \mu_{3\theta}\sin\frac{3\pi u}{2a} + \mu_{5\theta}\sin\frac{5\pi u}{2a}\right]$$

$$= \frac{520\times 3.675}{6.0}\,[0.02\times 1.0 + (-0.02)\,(-1.0) + (0)\,(1.0)]$$

= 320 (0.02 + 0.02) = 13 Kgm.

∴ Moment on the cross-beam = 13 × 6.0 = 78 Kgm.

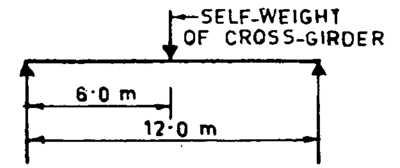


FIG. 6·14 LONGITUDINAL DISPOSITION OF CROSS-GIRDER

∴ Total dead load moment on the cross beam = 1320 + 2662 + 78 = 4060 Kgm.

Live load moment

In Illustrative Example 6.3, the live load moment on the cross-beam of the same deck has been determined for Class AA (tracked) loading. The deck under consideration is subjected to Class 70-R loading. Therefore, some modification is necessary in finding out the live load moment on the cross girder. Since θ and α values of both the decks are the same, the influence line for transverse moment co-efficients as shown in Fig. 6.10 will remain the same. However, since the length of Class 70-R tracked loading is 4.57 m. in place of 3.60 m. for Class AA tracked loading, the loading will be 7.66 tonnes/m. for the former in place of 9.72 tonnes/m. for the latter. Another modification is the use of Fig. B-15 in place of B-14 (Appendix B) for the determination of the values of $\sin\frac{n\pi}{2}\sin\frac{n\pi c}{2a}$.

μ_θ, $\mu_{3\theta}$ and $\mu_{5\theta}$ values are 0.16, (-)0.020 and 0.020 respectively as in Illustrative Example 6.3. The values of $\sin\frac{n\pi}{2}\sin\frac{n\pi c}{2a}$ are obtained from Fig. B-15 (Appendix B) and are 0.55, -1.00 and 0.15 for n = 1, 3 and 5 respectively for 2a = 12.0 m.

∴ Transverse moment per metre length

$$= \frac{4bw}{\pi}\left[\mu_\theta \sin\frac{\pi}{2}\sin\frac{\pi c}{2a} - \frac{\mu_{3\theta}}{3}\sin\frac{3\pi}{2}\sin\frac{3\pi c}{2a} + \frac{\mu_{5\theta}}{5}\sin\frac{5\pi}{2}\sin\frac{5\pi c}{2a}\right]$$

$$= \frac{4\times 3.675\times 7.66}{\pi}\left[0.16\times 0.55 - \frac{(-)0.020}{3}\times(-)\,1.00 + \frac{0.020}{5}\times 0.15\right]$$

$= 35.84\,(0.088 - 0.0067 + 0.0006) = 3584\times 0.08 = 2.87$ tm/m.

∴ Moment on the cross-beam = Transverse moment per metre length × spacing of cross-girder = 2.87 × 6.0 = 17.22 tm.

Moment on the cross-beam with 10 percent impact = 1.1 × 17.22 = 18.94 tm.

Due to local concentration of load, this moment may be increased by 10 percent.

∴ Design L.L.M. on cross girder = 1.1 × 18.94 = 20.83 tm. = 20,830 Kgm.

∴ Design moment = DLM + LLM = 4060 + 20,830 = 24,890 Kgm. = 2,44,000 Nm.

Design of section for cross-beam

Effective flange width shall be the least of the following :

i) ¼ span = ¼ spacing of outer girder
 = ¼ × 2 × 2.45 = 1.22 m.

ii) Spacing of cross beam = 6.0 m.

iii) Web thickness + 12 × flange thickness
 = 0.25 + 12 × 0.215 = 2.83 m.

Hence 1.22 m. width governs the design. From Fig. 8.15, moment of resistance of compression (average stress = 0.8 σ_c) = 1220 × 215 × 0.8 × 6.7 × 865 Nmn = 12,16,100 × 10^3 Nmm = 12,16,100 Nm > design moment of 2,44,000 Nm.

Hence depth is satisfactory.

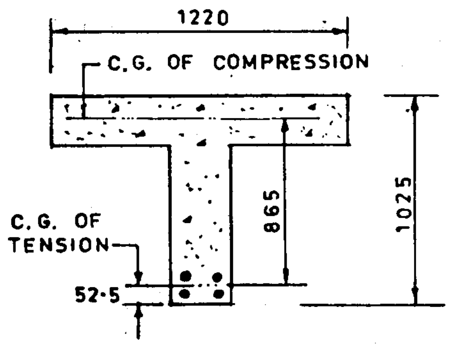
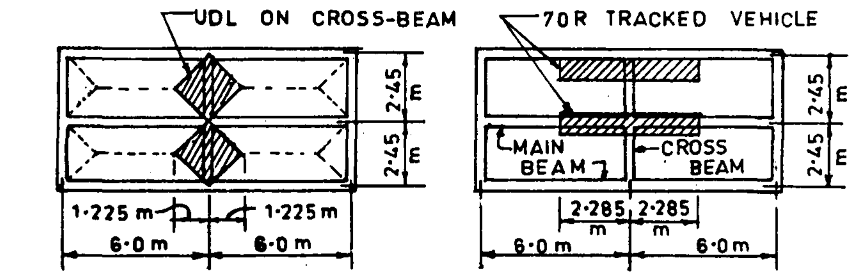


FIG. 8·15 EFFECTIVE SECTION OF CROSS-GIRDER

$$As = \frac{2{,}44{,}000\times 10^3}{195(av)\times 865} = 1447 \text{ mm}^2$$; Provide 4 Nos. 25 Φ HYSD bars (As = 1960 mm²)



(a) DISTRIBUTION OF DEAD LOAD UDL ON CROSS BEAM (b) DISTRIBUTION OF LIVE LOAD ON CROSS BEAM

FIG.8·16 DISTRIBUTION OF LOAD ON CROSS-GIRDER (Ex.8·1)

a) Dead load shear

The distribution of dead load of slab, wearing course etc. is shown in Fig. 8.16a.

i) Shear due to weight of deck slab and wearing course
$= 2 \times \frac{1}{2} \times 2.45 \times 1.225 \times (0.215 \times 2400 + 0.085 \times 2500) = 2186$ Kg.

ii) Shear due to self wt. of cross-beam $= \frac{1}{2} \times 2.45 \times 0.81 \times 0.25 \times 2400 = 595$ Kg.

iii) Weight of central girder per m. $= \frac{1}{3} \times 2776$ Kg. (vide dead load calculation for the design of girder)
$= 925$ Kg.

Shear due to wt. of central girder $= \dfrac{925 \times 12.0}{4} = 2775$ Kg.

$\therefore$ Total dead load shear $= 2186 + 595 + 2775 = 5556$ Kg.

b) Live load shear

Class 70-R tracked vehicle will produce maximum shear when the load is placed on the deck as shown in Fig. 8.16b.

Longitudinal Distribution :

Reaction of the tank load on the cross-beam (assuming simple reaction) $= \dfrac{2 \times 35.0 \times 4.858}{6.0} = 56.67$ tonnes.

Transverse Distribution :

The portion of the load coming on the cross girder after longitudinal distribution will be shared by the main beams in proportion to the distribution co-efficients already found out previously. The reaction on the outer girder will give the shear on the cross beam.

Reaction on outer girder $= \dfrac{56.67}{3} \times 1.45$ (distribution co-efficient) $= 27.39$ tonnes $= 27{,}390$ Kgs.

$\therefore$ Design shear on the cross-beam = D.L. shear + L.L. shear $= 5556 + 27{,}390 = 32{,}946$ Kg. $= 3{,}22{,}900$ N.

Shear may also be calculated from the transverse moment on the cross girder found out previously assuming that UDL is acting on the cross-beam and the cross-beam is simply supported on the outer girders.

Design transverse moment = DLM + LLM = 4060 + 20,830 = 24,890 Kgm.

If w is the UDL, $\frac{wL^2}{8} = 24{,}890$ Kgm. $\therefore w = \frac{24{,}890 \times 8}{(L)^2} = \frac{24{,}890 \times 8}{(4.9)^2} = 8293$ Kg.

$\therefore$ Design shear $= \frac{wL}{2} = \frac{8293 \times 4.9}{2} = 20{,}674$ Kg. = 2,02,600 N.

The higher shear value of 3,22,900 N determined by the other method is considered in the design.

Shear stress $= \frac{V}{bd} = \frac{3{,}22{,}900}{250 \times 922.5} = 1.40$ MPa

Since the shear stress exceeds the permissible limit of 0.34 MPa without shear reinforcement, the same is necessary. Permissible shear with shear reinforcement for M20 grade concrete = 0.07 × 20 = 1.40 MPa.

Shear Reinforcement :

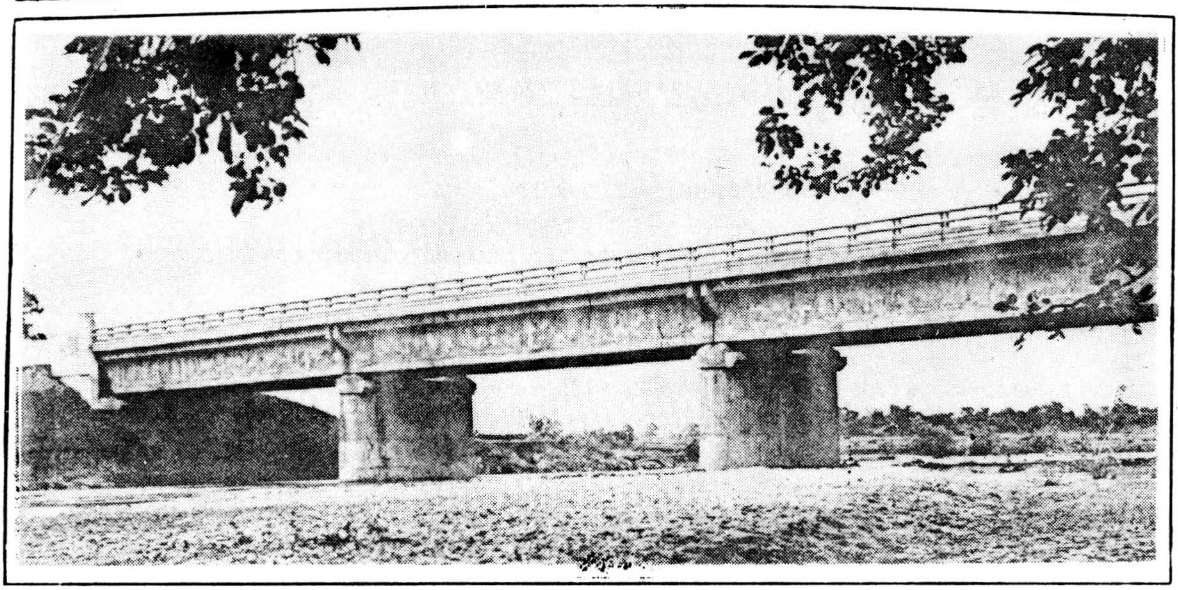
Using 2 nos. 25 Φ HYSD bars bent up bars, shear resistance = 2 × 490 × 200 × 0.707 = 1,38,600 N. Balance shear of 1,84,300 N is to be resisted by stirrups. Using 10 Φ 2 legged stirrups @ 125 mm., Asw required $= \frac{Vs}{\sigma_s d} = \frac{1{,}84{,}300 \times 125}{200 \times 922.5} = 125$ mm². Asw provided = 2 × 78 = 156 mm². Hence satisfactory.

8.4 DETAILS OF FEW SLAB AND GIRDER BRIDGES

8.4.1 The Ministry of Shipping and Transport (Roads Wing), Govt. of India has published **"Standard Plans for Highway Bridges — Concrete T-beam Bridges"** with 7.5 m. carriage way and with or without footpaths. The bridge decks have three number T-beams of varying depths depending upon spans. However, there are three number cross-girder for effective spans upto 16.5 m. and four number cross-girder for effective spans of 18.75 to 24.75 m. The design is based on M20 grade concrete and S 415 grade steel. Important details of these bridges are given in Table 8.1 and 8.2. For more details, the aforesaid plans may be consulted.

TABLE 8.1 : DETAILS OF SOME T-BEAM BRIDGES WITH 1.5m WIDE FOOTPATHS *(MOS & T)* [2]

Sl. No.	Effective Span (m)	Overall depth of T-beam (mm)	Overall depth of cross-beam (mm)	Overall depth of deck slab (mm)	Reinforcement (HYSD bars) in				
					T-beam		Cross beam		Deck Slab
					Outer	Central	End	Inter-mediate	
1	10.5	1100	800	240	16-25 Φ	16-25 Φ	3-25 Φ 2-20 Φ	6-25 Φ	12 Φ @ 115
2	12.5	1300	800	240	12-28 Φ 4-25 Φ	14-28 Φ	3-25 Φ 2-20 Φ	3-28 Φ 3-25 Φ	12 Φ @ 105
3	14.5	1600	1300	240	16-28 Φ	14-28 Φ	5-20 Φ	8-25 Φ	12 Φ @ 100
4	16.5	1700	1300	240	18-28 Φ	16-28 Φ	5-25 Φ	8-25 Φ	12 Φ @ 100
5	18.75	1900	1300	240	22-28 Φ	20-28 Φ	4-16 Φ 4-12 Φ	6-25 Φ	12 Φ @ 105
6	21.75	2200	1300	240	24-28 Φ	22-28 Φ	4-16 Φ 4-12 Φ	7-25 Φ	12 Φ @ 100
7	24.75	2500	1300	240	26-28 Φ	24-28 Φ	4-16 Φ 4-12 Φ	7-25 Φ	12 Φ @ 100



Photograph 2
BRIDGE OVER KUMARI RIVER
(R. C. Slab and Girder Bridge on N. H. 32 in the District of Purulia, West Bengal. The bridge consists of 3 spans of 21.0 m having a total length of 63.0 m) *(Highway Bridges)*[4]

TABLE 8.2 : DETAILS OF SOME T-BEAM BRIDGES WITHOUT FOOTPATHS *(MOS & T)*[2]

Sl. No.	Effective Span (m)	Overall depth of T-beam (mm)	Overall depth of cross beam (mm)	Overall depth of deck slab (mm)	Reinforcement (HYSD bars) in				
					T-beam		Cross-beam		Deck Slab
					Outer	Central	End	Inter-mediate	
1	10.5	1075	775	215	16-25 Φ	14-25 Φ	3-25 Φ 2-20 Φ	7-25 Φ	12 Φ @ 110
2	12.5	1275	775	215	14-28 Φ	12-28 Φ	3-25 Φ 2-20 Φ	8-25 Φ	12 Φ @ 110
3	14.5	1575	1275	215	14-28 Φ	12-28 Φ	5-20 Φ	8-22 Φ	12 Φ @ 110
4	16.5	1675	1275	215	16-28 Φ	14-28 Φ	5-20 Φ	7-25 Φ	12 Φ @ 110
5	18.75	1875	1275	215	20-28 Φ	16-28 Φ	4-16 Φ 4-12 Φ	8-20 Φ	12 Φ @ 110
6	21.75	2175	1275	215	22-28 Φ	18-28 Φ	4-16 Φ 4-12 Φ	7-25 Φ	12 Φ @ 110
7	24.75	2475	1275	215	24-28 Φ	20-28 Φ	4-16 Φ 4-12 Φ	7-25 Φ	12 Φ @ 110

8.5 REFERENCES

1. IRC : 21-1987 — Standard Specifications and Code of Practice for Road Bridge Section III — Cement Concrete (Plain and Reinforced) (Second Revision), Indian Roads Congress.
2. Ministry of Shipping and Transport (Roads Wing) : Standard Plans for Highway Bridges, Volume III, Concrete T-beam Bridges, 1983.
3. IS : 456-1978 — Code of Practice for Plain and Reinforced Concrete (Third Revision), Indian Standards Institution.
4. *"Highway Bridges in West Bengal"* — Published by the Public Works and Public Works (Roads) Departments, Govt. of West Bengal, 1977
5. Dayaratnam, P — *"Design of Reinforced Concrete Structures (Second Edition)"* — Oxford & IBH Publishing Co.
6. Legat, A.W., Dunn, G. & Fairhurst, W.A. — *"Design and Construction of Reinforced Concrete Bridges"* — Concrete Publications Ltd., 14, Dartmouth Street, London, S.W.-1.
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8. Rakshit, K.S. — *"Load Distribution in Bridge Decks : A Comparative Study"* — Construction Engineers of India, 1966. State Engineers' Association, Calcutta.
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Chapter 9

SKEW AND CURVED BRIDGES

9.1 SKEW BRIDGES

9.1.1 The behaviour of skew bridges differ widely from that of normal bridges and therefore, the design of skew bridges needs special attention. In normal bridges, the deck slab is perpendicular to the supports and as such the load placed on the deck slab is transferred to the supports which are placed normal to the slab. Load transferance from a skew slab bridge, on the otherhand, is a complicated problem because there remains always a doubt as to the direction in which the slab will span and the manner in which the load will be transferred to the support.

FIG. 9·1 PLAN OF SKEW DECK SLAB

9.1.2 It is believed that the load travels to the support in proportion to the rigidity of the various paths and since the thickness of the slab is the same everywhere, the rigidity will be maximum along shortest span i.e. along the span normal to the faces of the piers or abutments. In Fig. 9.1, though the span of the deck is the length BC or DE, the slab will span along AB or CD being the shortest distance between the supports. Therefore, the plane of maximum stresses in a skew slab are not parallel to the centre line of roadway and the deflection of such slab produces a warped surface.

9.1.3 The effect of skew in deck slabs having skew angles upto 20 degrees, is not so significant and in designing such bridges, the length parallel to the centre line of the roadway is taken as the span. The thickness of the slab and the reinforcement are calculated with this span lengths and the reinforcement are placed parallel to the centre line of the roadway. The distribution bars are, however, placed parallel to the supports as usual. When the skew angle varies from 20 degrees to 50 degrees, the skew effect becomes significant and the slab tends to span normal to the supports as explained in Art. 9.1.1. In such cases, the slab thickness is determined with shortest span but the reinforcement worked out on the basis of shortest span are multiplied by Sec. $^2\theta$ (θ

being the skew angle) and are placed parallel to the roadway as shown in Fig. 9.2a, the distribution bars being placed parallel to the supports as usual.

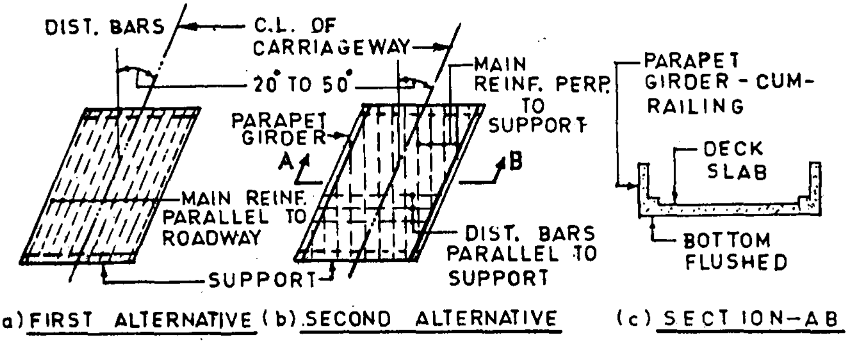


FIG.9·2 ARRANGEMENT OF REINFORCEMENT IN SLAB WITH SKEW ANGLE FROM 20 TO 50 DEGREES

9.1.4 It is also a common practice to place the reinforcement perpendicular to the support when the skew angle lies between 20 degrees to 50 degrees. The thickness and the reinforcement are determined with span normal to the support but since in placing the reinforcement perpendicular to the supports, the corner reinforcement within the area ABF or CDE (Fig. 9.1) do not get any support on one side to rest on, the slab below the footpath (for bridge with footpath) or below the road kerb (for bridge without footpath) shall be provided with extra reinforcement to act as concealed beam. Alternatively, parapet girders as illustrated in Fig. 9.2b and 9.2c may also be provided along the edge of the slab. Such parapet girders are made flush with the bottom of the slab and extended above the slab to the required height to form the solid parapet. This sort of deck requires less quantity of steel in slabs but parapet girders need additional cost.

9.1.5 For bridges of skew angles more than 50 degrees, girders should be used even though the spans are comparatively less. Where the width of the bridge is not much, the girders may be placed parallel to the roadway and the slab thickness and the reinforcement may be designed with the spacing of the girders as the span. The reinforcement are placed normal to the girders (Fig. 9.3a). In wider multi-lane skew crossings with large skew angles, however, it is preferable to use the girders at right angles to the supports. In such cases again, the

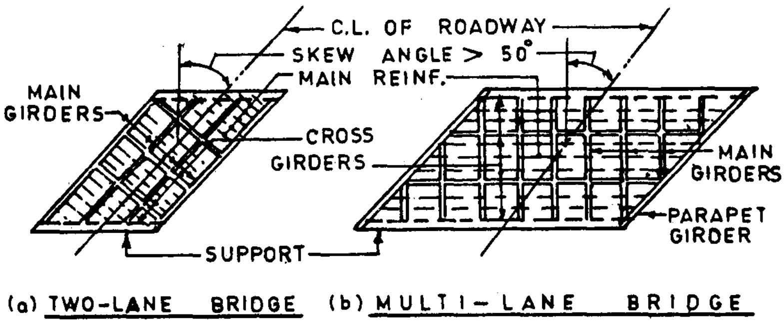


FIG. 9·3 GIRDER BRIDGES WITH SKEW ANGLE GREATER THAN 50 DEGREES triangular portions need parapet girders to support one end of the girders. The reinforcement are used normal to the girders as shown in Fig. 9.3b.

9.1.6 Reaction at Support

It has been observed that due to the effect of skew, the reactions at supports are not equal but the same is more at obtuse angle corners and less at acute angle corners depending on the angle of skew. For skews upto 20 degrees, the increase in the reaction on the obtuse angle corners is zero to 50 percent and for skews from 20 degrees to 50 degrees, the increase is from 50 percent to 90 percent of the average reaction. The reaction on the obtuse angle corner becomes twice the average reaction thus making the acute angle corner a zero presure point when the skew angle reaches about 60 degrees.

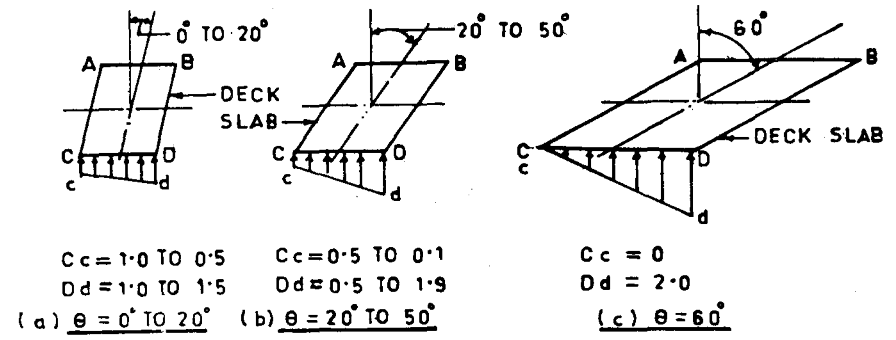


FIG. 9·4 VARIATION OF REACTION ON SUPPORTS FOR SLABS HAVING VARIOUS ANGLES OF SKEW (CAI)[2]

9.1.7 Creep Effect

9.1.7.1 Observations reveal that the longer diagonal of the skew deck connecting the acute angle corners has a tendency to elongate due possibly to the nature of the load transference on the supports resulting in the movement or creep of the acute angle corners as illustrated in Fig. 9.5a. This creeping effect of the deck slab induces tension along longer diagonal and tension cracks may appear if sufficient steel is not provided to cater for this tensile stress (vide Fig. 9.5b). Also on account of the creep, lifting and consequential cracks occur at

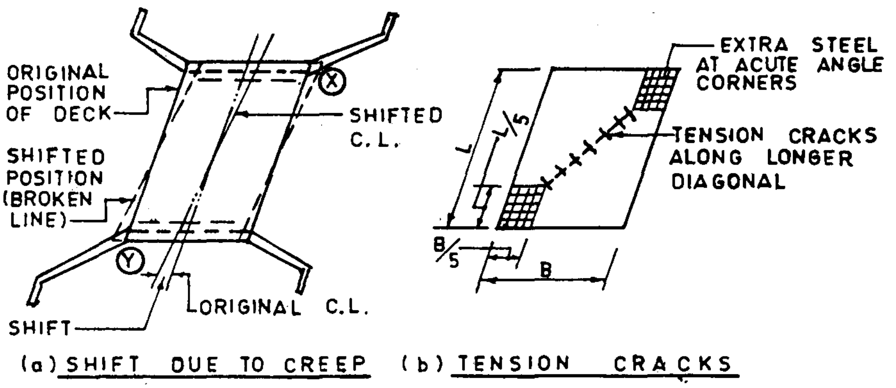


FIG. 9·5 CREEP IN SKEW BRIDGES (CAI)[2]

the acute angle corners and additional steel requires to be provided at the top in both directions to prevent crack due to lifting of the corners.

9.1.7.2 It may be seen in Fig. 9.5a that due to the creep of the deck slab, considerable thrust is induced on the wing walls at X and Y i.e. at the junction of abutment and the wing wall resulting in development of cracks in wing walls or heavy damage. In order to avoid the damage to wing walls due to creep effect, it has been suggested by some authorities to provide fixed bearings over abutments instead of free bearings so that movement of the deck due to creep effect is prevented over the abutments.

9.1.7.3 Sometimes the deck slab is fixed to the abutment cap with dowel bars which seems to be the most effective means of guarding against the creep effect. Creep may be stopped over piers by providing some raised blocks or buffers over piers. This arrangement is shown in Fig. 9.6.

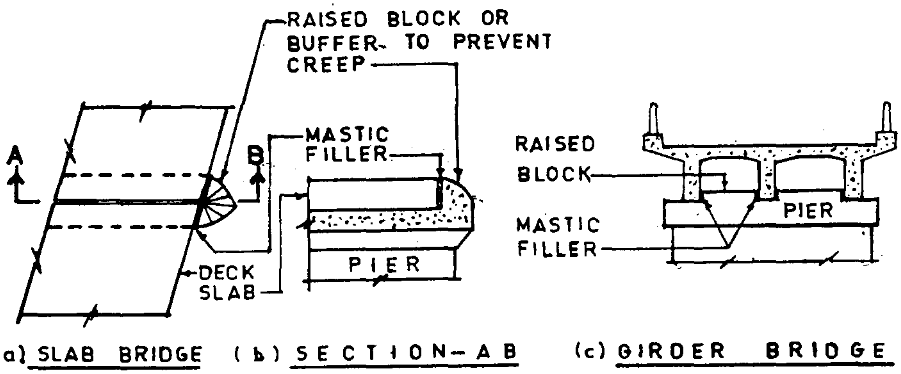


FIG. 9·6 PREVENTION OF CREEP

9.1.8 LAYOUT OF BEARINGS

9.1.8.1 As explained earlier, preventive measure should be taken to guard against the movement of the deck due to creep. It is suggested that the following steps, if taken, may produce the desired result.

i) Up to 15.0 m span for a single span bridge fixed bearings on both the abutments may be used. The construction of single span concrete bridges with two fixed bearings has been used for years by the Wisconsin Highway Commission for span lengths up to 45 feet (13.72 m). None of these bridges showed signs of creep(2).

ii) For multi-span simply supported bridges, fixed bearings over the abutments and free or fixed bearings over the piers. With this arrangement, it may be necessary to use two free bearings on one pier.

9.1.8.2 The layout of the bearings should be such that no obstruction is created against the free movement of the expansion bearings. This requires the bearings to be oriented at right angles to the girders instead of parallel to the piers or abutments (similar to the normal crossings). The typical layouts of the bearings in skew bridges are indicated in Fig. 9.7.

9.1.9 LAYOUT OF EXPANSION JOINTS S

9.1.9.1 The main difference in the various types of layout illustrated in Fig. 9.7 is in the manner of providing the expansion joint between the adjacent decks. For getting straight expansion joint, the type shown in Fig. 9.7a is adopted but it requires more pier width since some space between the bearings of the adjacent spans remains unused. The type of Fig. 9.7b also gives a straight joint but in order to reduce the width of pier, the bearings are to be brought closer. This necessiates encroachment of deck on the girders of the adjacent spans

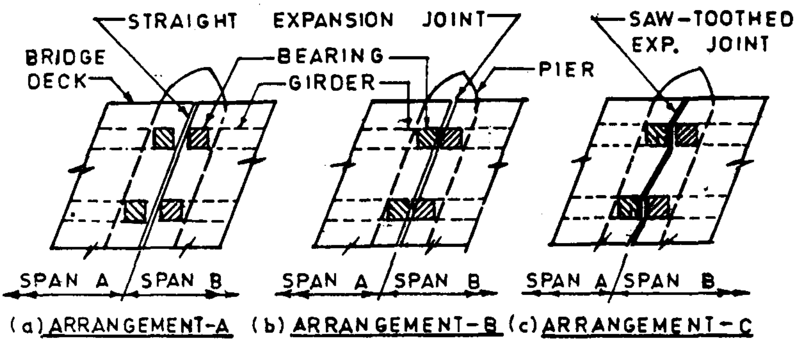


FIG. 9·7 LAYOUT OF BEARINGS AND EXPANSION JOINTS IN SKEW GIRDER BRIDGES (CAI)[2]

which is achieved by making a notch over the affected portions of the girders and the deck slab rests on these notches. Suitable joint filler like lead sheet or tarred paper may be inserted between the girders and the deck slab for free movement of the expansion joint. The width of pier as well as the location of the bearings for the type shown in Fig. 9.7c are the same as in Fig. 9.7b but a saw-toothed type of expansion joint is adopted here with a view to avoid the sort of arrangements necessary for the second one. Each of the types described herein has certain merits and demerits and the one most suited for the bridge under consideration may be used.

9.1.10 The major points which a designer has to consider carefully in the design of skew bridges have been described here very briefly. Now to illustrate the design principles, one worked out example is presented below.

ILLUSTRATIVE EXAMPLE 9.1

Design a solid slab skew bridge having a clear span of 7.5 m along the roadway without any footpath and a skew angle of 25 degrees with IRC loading for N.H. Standard. M20 grade concrete and S415 grade steel will be used.

Solution

Since the skew angle exceeds 20 degrees, the slab thickness may be designed with span normal to the support and the reinforcement worked out with this span may be multiplied by Sec. $^2\theta$ and the same may be provided parallel to the roadway as stated in Art. 9.1.3.

Clear span normal to the supports = 7.5 cos 25° = 7.5 × 0.9063 = 6.80 m

Effective span = Clear span + effective depth

Assuming an overall slab thickness of 600 mm, effective depth is 600 – 40 = 560 mm. = 0.56 m.

∴ Effective span = 6.80 + 0.56 = 7.36 m.

Dead load moment

Load per sq.metre of deck —

i)	600 mm slab @ 2400 Kg/m^3	1440 Kg.
ii)	85 mm thick average wearing coarse @ 2500 Kg/m^3	213 Kg.
iii)	For wheel guard, parapet kerb, railings etc. (same as those for solid slab bridges — Chapter 7)	140 Kg.
		1793 Kg.
	Say	1800 Kg.

$\therefore$ Dead load moment per metre width = $\dfrac{1800 \times (7.36)^2}{8}$ = 12,190 Kgm.

Live load moment

Single lane of Class 70-R tracked vehicle when placed centrally will produce maximum moment.

Dispersion across span : $\dfrac{b}{L} = \dfrac{8.43 \sec\theta}{7.36} = \dfrac{9.30}{7.36} = 1.26$

$\therefore$ K from Table 5.2 for simply supported slab = 2.69

$$b_e = Kx\left(1 - \frac{x}{L}\right) + W = 2.69 \times 3.68\left[1 - \frac{3.68}{7.36}\right] + (0.84 + 2 \times 0.085)$$

$= 2.69 \times 3.68 \times 0.5 + 1.01 = 4.95 + 1.01 = 5.96$ m.

Hence overlap occurs.

Placing the track at the extreme end (i.e. 1.2 m from road kerb)

$b_e = \frac{1}{2} \times 5.96 + 2.04 + (0.42 + 1.2 + 0.465) = 7.105$ m

Dispersion along span = 4.57 + 2(0.6 + 0.085) = 5.94 m.

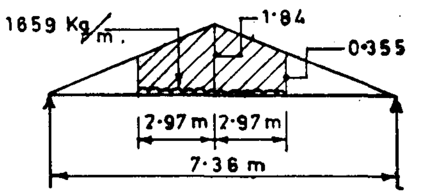


FIG. 9·8 (Example 9·1)

\Intensity of load $= \dfrac{70{,}000}{7.105 \times 5.94} = 1659$ Kg/m^2

$\therefore$ Live load moment per metre width = $\dfrac{2 \times (1.84 + 0.355)}{2} \times 2.97 \times 1659$ = 10,815 Kgm.

Impact factor = 25 percent

$\therefore$ L.L.M. with impact = 1.25 × 10,815 = 13,520 Kgm.

$\therefore$ Design moment = DLM + LLM = 12,190 + 13,520 = 25,710 Kgm. = 2,52,000 Nm.

$$\therefore d = \sqrt{\frac{2{,}52{,}000 \times 10^3}{0.95 \times 10^3}} = 515 \text{ mm}$$

Overall depth assumed = 600 mm.

$\therefore$ Effective depth provided = 600 – 30 (cover) – 10 ($^1/_2$ dia of bar) = 560 mm.

Hence depth is satisfactory.

$$As = \frac{2{,}52{,}000 \times 10^3}{200 \times 560 \times 0.904} = 2490 \text{ mm}^2$$

Provide 22 Φ HYSD bars @ 150 (As = 2535 mm^2)

If the main reinforcement is intended to be provided in the deck slab parallel to the roadway as in Fig. 9.2a, area of reinforcement is equal to As. Sec$^2\theta$ = 2490 (Sec. 25°)^2 = 2490 × 1.22 = 3038 mm^2

Use 22 Φ main bars @ 125 mm (As = 3040 mm^2)

Distribution Steel

Distribution steel may be calculated on the same principle as in the case of design of square crossing solid slab bridge vide Illustrative Example 7.1.

Moment in the transverse direction = 0.3 LLM + 0.2 other moments = 0.3 × 13,520 + 0.2 × 12,190 = 6494 Kgm. = 63,600 Nm.

$$\therefore \text{ As} = \frac{63{,}600 \times 10^3}{200 \times 543 \times 0.904} = 648 \text{ mm}^2$$

Adopt 12 Φ HYSD bars @ 150 (As = 753 mm^2)

Shear and Bond Stress

The increase of support reaction near obtuse angle corner shall be duly considered in working out the shear and bond stresses.

Since the skew angle is 25 degrees, the maximum reaction at the obtuse angle corner may be taken as 1.55 times the normal reaction (vide Fig. 9.4). Average increased value for the half width of the deck may be taken as 1.30 times the normal reaction.

$$\therefore \text{ Maximum D.L. Shear per metre width } = \frac{1800 \times 7.36}{2} \times 1.30 = 8610 \text{ Kg.}$$

Live Load Shear

Dispersion (at $x = \frac{5.94}{2}$ m),

$$b_e = Kx\left(1 - \frac{x}{L}\right) + W = 2.69 \times 2.97 \times 0.6 \times 1.01 = 5.80 \text{ m.}$$

Considering the effect of two tracks,

$$b_e = \tfrac{1}{2} \times 5.80 + 2.04 + (0.42 + 1.2 + 0.465) = 7.045 \text{ m}$$

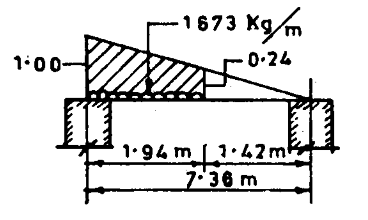


FIG. 9·9 (Example 9·1)

$$\text{Intensity of live load } = \frac{70{,}000}{5.94 \times 7.045} = 1673 \text{ Kg/m}^2$$

L.L. Shear = Area of influence line diag × intensity of loading

$= \tfrac{1}{2}(1.0 + 0.24) \times 5.94 \times 1673 = 6160$ Kg.

L.L. Shear with impact = 6160 ×1.25 = 7700 Kg.

L.L. Shear due to 30 percent increased value for the half width near the obtuse angle corner = 1.3 × 7700 = 10,010 Kg.

∴ Design Shear = DL Shear + LL Shear = 8610 + 10,010 = 18,620 Kg = 1,82,500 N.

$$\therefore \text{ ShearStress} = \frac{V}{bd} = \frac{1{,}82{,}500}{1000 \times 560} = 0.33 \text{ MP}_a$$

Permissible value without shear reinforcement as per Table 5.12 = 0.34 MP$_a$. Hence safe.

9.1.11 Arrangement of Reinforcement

9.1.11.1 Two types of arrangement of reinforcement in line with Art. 9.1.3 and 9.1.4 are shown in Fig. 9.10 and 9.11 respectively. Reinforcement at top of acute angle corners are provided to prevent cracks due to lifting of the acute angle corners as mentioned in Art. 9.1.7.

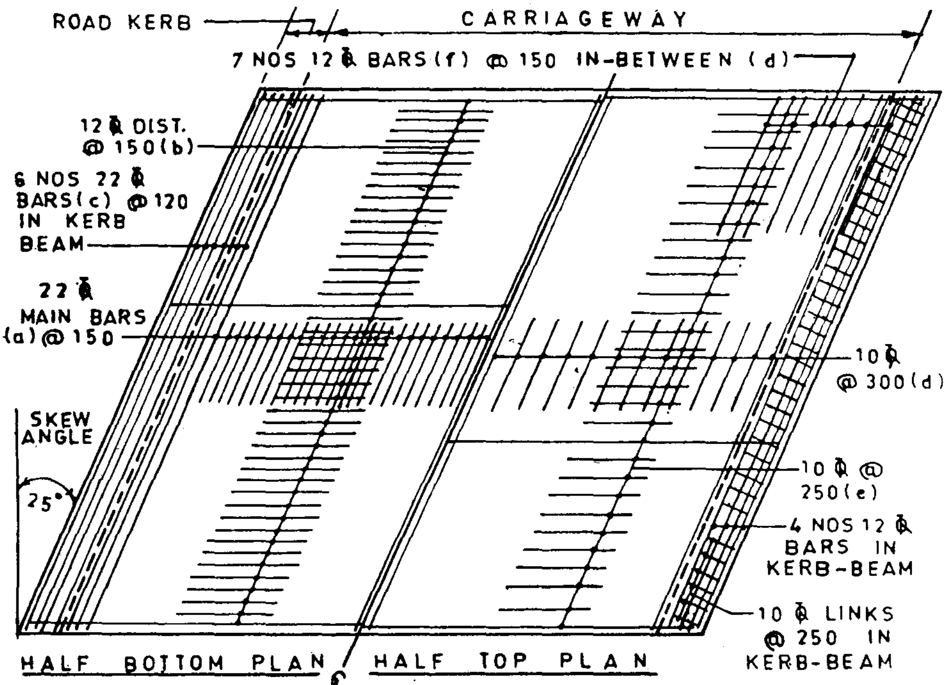


FIG. 9·10 REINFORCEMENT DETAILS OF SKEW DECK SLAB
(Main reinforcement parallel to carriageway)

9.1.11.2 As calculated before, the area of main reinforcement, if placed perpendicular to the support, is 2490 mm^2 in which case 22 Φ @ 150 mm gives As = 2535 mm^2. However, if the reinforcement is placed parallel to the roadway, area of steel required = 3038 mm^2 for which 22 Φ @ 125 mm is required to be provided (As = 3040 mm^2).

9.1.12 Details of Few Skew Slab Bridges

9.1.12.1 The Ministry of Shipping and Transport, (Roads Wing), Govt. of India, has published ***"Standard Plans for Highway Bridges — Vol. II — Concrete Slab Bridges"*** with 7.5 m carriageway and with or without footpath. The spans (effective right span at right angles to the supports) for which the details are available are 4.37 m, 5.37 m, 6.37 m and 8.37 m with skew angles of 15°, 30°, 45° and 60° for each span. The design is based on M20 grade concrete and S415 grade steel. Salient features of these skew bridges are given in Table 9.1 and 9.2. For further details, the standard plans under reference may be referred.

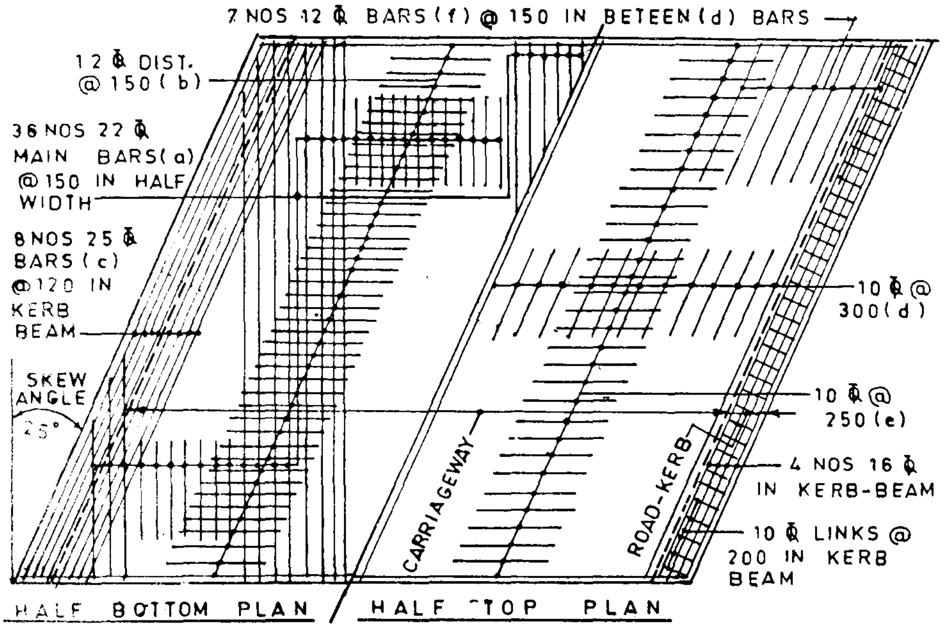


FIG. 9·11 REINFORCEMENT DETAILS OF SKEW DECK SLAB
(Main reinforcement perpendicular to support)

TABLE 9.1 : DETAILS OF SOME SKEW SLAB BRIDGES WITH 1.5m WIDE FOOTPATHS (MOS & T)[1]

Effective right span (m)	Overall length of slab (m)	Overall depth of slab (mm)	Angle of Skew, θ	Bottom Steel		Top Steel		Stiffening Steel under footpath
				Main	Distributor	Longitudinal	Transverse	
4.37	4.74	405	15°	16 Φ @125	10 Φ @130	10 Φ @300	10 Φ @300	16 Φ 22 Nos.
			30°	Do	Do	Do	Do	16 Φ 22 Nos.
			45°	Do	Do	Do	10 Φ @180	16 Φ 28 Nos.
			60°	Do	Do	Do	10 Φ @130	16 Φ 30 Nos.
6.37	6.74	470	15°	20 Φ @125	10 Φ @85	Do	12 Φ @300	20 Φ 22 Nos.
			30°	Do	Do	Do	Do	20 Φ 22 Nos.
			45°	Do	Do	Do	12 Φ @180	20 Φ 28 Nos.
			60°	Do	Do	Do	12 Φ @110	20 Φ 30 Nos.
8.37	8.74	630	15°	25 Φ @140	Do	Do	16 Φ @300	25 Φ 20 Nos.
			30°	Do	Do	Do	Do	25 Φ 20 Nos.
			45°	Do	Do	Do	16 Φ @200	25 Φ 22 Nos.
			60°	Do	Do	Do	16 Φ @130	25 Φ 28 Nos.

TABLE 9.2 : DETAILS OF SOME SKEW SLAB BRIDGES WITHOUT FOOTPATHS (MOS & T)[1]

Effective right span (m)	Overall length of slab (m)	Overall depth of slab (mm)	Angle of Skew, θ	Bottom Steel		Top Steel		Stiffening Steel under road kerb
				Main	Distributor	Longitudinal	Transverse	
5.37	5.74	465	15°	20 Φ @130	10 Φ @90	10 Φ @300	10 Φ @300	20 Φ 16 Nos.
			30°	Do	Do	Do	Do	Do
			45°	Do	Do	Do	10 Φ @200	20 Φ 18 Nos.
			60°	20 Φ @120	10 Φ @110	Do	10 Φ @180	Do
6.37	6.74	540	15°	20 Φ @115	10 Φ @100	Do	10 Φ @300	20 Φ 20 Nos.
			30°	Do	Do	Do	10 Φ @260	Do
			45°	Do	Do	Do	10 Φ @200	Do
			60°	20 Φ @95	10 Φ @160	Do	10 Φ @160.	Do
8.37	8.74	700	15°	25 Φ @150	10 Φ @90	Do	10 Φ @260	25 Φ 14 Nos.
			30°	Do	10 Φ @110	Do	10 Φ @230	25 Φ 16 Nos.
			45°	Do	10 Φ @150	Do	10 Φ @160	Do
			60°	25 Φ @95	10 Φ @180	Do	10 Φ @90	25 Φ 18 Nos.

9.2 CURVED BRIDGES

9.2.1 Curved bridges are normally provided for viaducts and interchanges where divergent traffic lanes are converged into a multi-lane bridge or overbridge and vice versa. One such example is the Second Hooghly Bridge at Calcutta with six-lane divided carriageway on the main bridge over the river and on the approach viaducts on both Calcutta and Howrah side. The interchanges on both Calcutta and Howrah side consist of a number of single or dual lane arms. A part of the Calcutta end viaduct and some of the arms of Calcutta and Howrah side interchanges are situated on curves as shown in Fig. 9.12.

Curved bridges over channels are sometimes required to be constructed when constraint of land inside a town or a city is such that constuction of such a bridge is the only possibility. Another situation where curved bridge has to be constructed has been described, in Art. 2.1.3 (ix) (Chapter 2).

9.2.2 Type of Piers

9.2.2.1 Selection of type of peirs for viaduct and interchange curved bridges is not a problem except in cases where traffic lanes are situated below. When the traffic lanes are located below the viaduct or interchange structures or where the bridge is constructed over a channel, the normal rectangular pier affects flow of traffic in case of the former and flow of water in case of the later (Fig. 9.13a). Therefore, under such circumstances, circular pier either solid or hollow, with pier cap above at right angles to the axis of the bridge is the right solution (Fig. 9.13b) in which case the flow will be smooth.

9.2.3 Layout of Bearings

9.2.3.1. The axis of a bridge deck for a curved bridge is not a straight line and changes direction at every point and for this reason, the pier or abutment caps supporting the deck through the bearings are not parallel to each

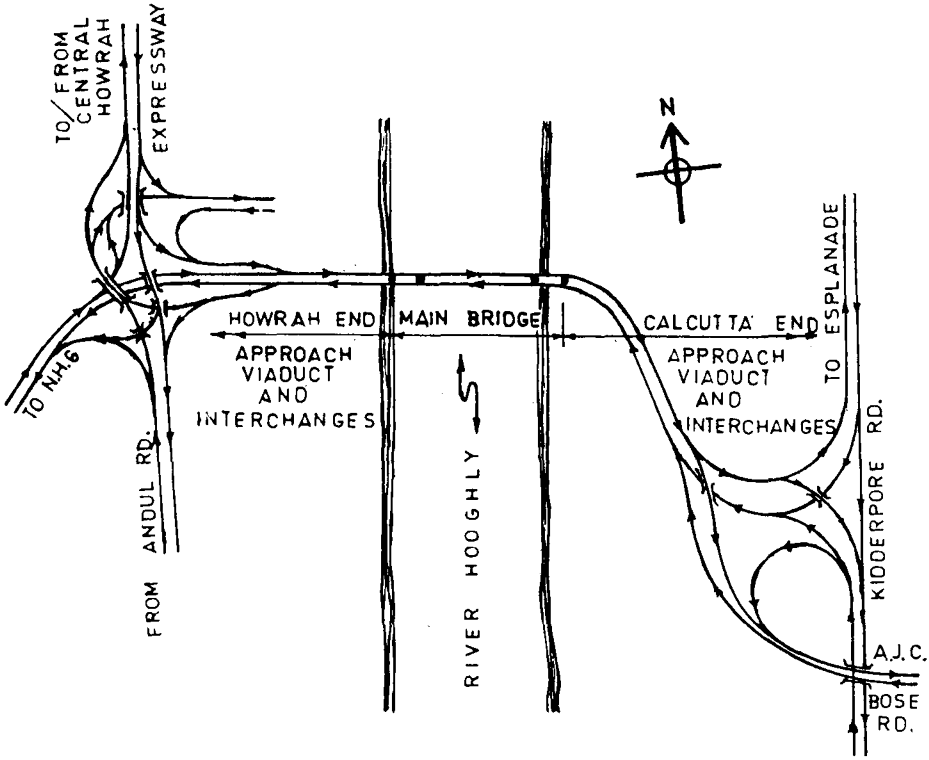
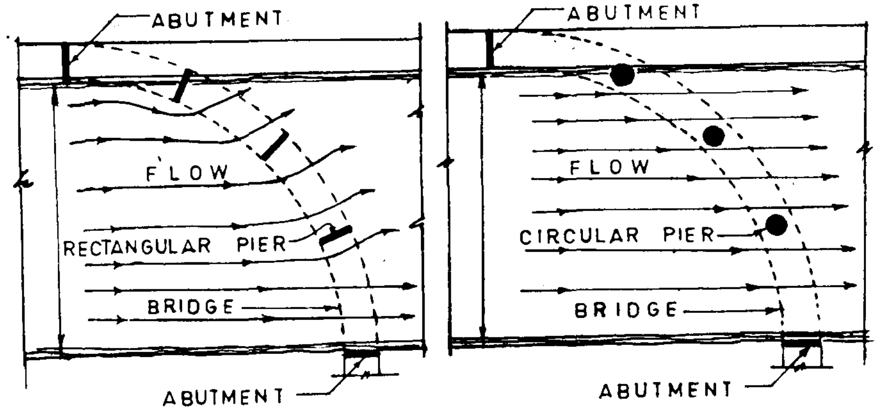


FIG. 9·12 SECOND HOOGHLY BRIDGE CALCUTTA WITH APPROACH VIADUCT AND INTERCHANGES



(a) CURVED BRIDGE WITH RECT. PIER
(Not Recommended in Channels)

(b) CURVED BRIDGE WITH CIRCULAR PIER
(Recommended in Channels)

FIG. 9·13 TYPES OF PIERS FOR CURVED BRIDGES other although these are at right angle to the axis of the bridge at these locations. But since the axis of the bridge changes direction from one pier cap to the other, it requries careful consideration in respect of fixing the axis of the metallic bearings, whether roller, rocker, hinged or sliding, although no such problem would normally arise in respect of elastomeric bearings or rubber pot bearings which are free to move in any direction and allow free horizontal movement and rotation of the superstructure.

9.2.3.2 The orientation of the free metallic bearings should be such that the direction of translation of the bearings shall coincide with the direction of movement of the bridge deck. The axis of a curved bridge changes direction at every point and hence the axis of the bridge at two adjacent piers is not the same. Therefore, it is to be decided in which manner the axis of the bearings shall be placed, whether at right angle to the bridge axis at such location or whether parallel to the pier-cap axis or in any other direction such that free movement of the deck due to temperature variation is allowed without any obstruction.

9.2.3.3 The direction of movement of a curved bridge deck at the free bearings can be found theoretically from Fig. 9.14 vide Art. 9.2.3.4.

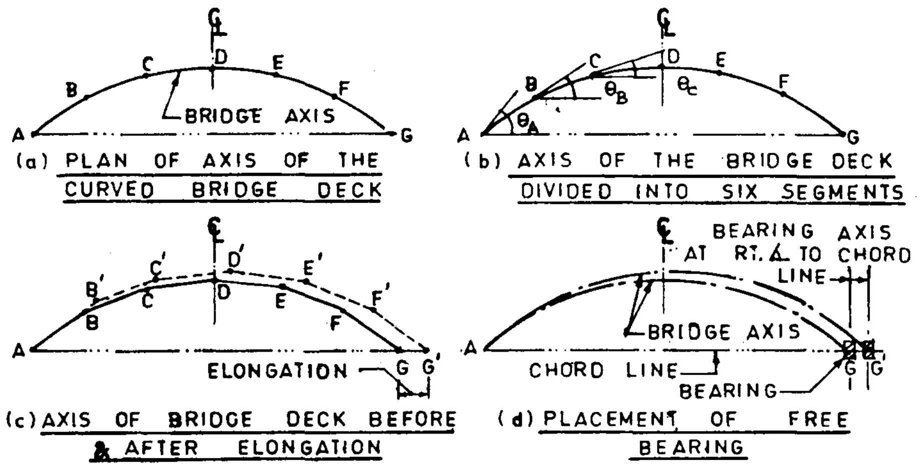


FIG. 9·14 LAYOUT OF FREE BEARING IN CURVED BRIDGES
(Rokshit)[4]

9.2.3.4 The curved bridge deck AG is divided into six equal segemnts, AB, BC, CD etc. and these lengths may be considered as equal to the chord lengths AB, BC, CD etc. specially when the number of division are large. Let length of these chords be equal to "1" and change in length due to temperature increase be "δl". Therefore, all the chords AB, BC, CD etc. get increased by δl tangentially. These increased lengths may be resolved into two perpendicular directions viz. along AG and perpendicular to AG. Increase in length of AB, BC, CD along AG direction is $\delta l\cos\theta_A$, $\delta l\cos\theta_B$, $\delta l\cos\theta_C$ respectively and increase of AB, BC, CD along perpendicular direction (outwards) is $\delta l\sin\theta_A$, $\delta l\sin\theta_B$, $\delta l\sin\theta_C$ respectively. Similarly, increase in length of DE, EF, FG along AG is $\delta l\cos\theta_E$, $\delta l\cos\theta_F$, $\delta l\cos\theta_G$ and along perpendicular direction (inwards) is $\delta l\sin\theta_E$, $\delta l\sin\theta_F$, $\delta l\sin\theta_G$ respectively. But since $\theta_A = \theta_G$, $\theta_B = \theta_F$ and $\theta_C = \theta_E$ and summation of the $\delta l\sin\theta$ of the left half is outwards and summation of the $\delta l\sin\theta$ of the right half is inwards, these outward and inward movements balance and the nett movement in the perpendicular direction is zero. Therefore, the movement of the curved bridge deck AG due to temperature variation will be along AG i.e. the chord line joining the axis of the bridge from one pier to the other and the nett movement will be $\Sigma\delta l\cos\theta$. Hence the bearing axis shall be at right angles to the chord line AG as shown in Fig. 9.14d. However, when elastomeric bearings are used, no such consideration need be made since these bearings are free to move in any direction.

9.2.4 Reactions at Piers

9.2.4.1 Fig. 9.15 shows the plan of a curved bridge deck. Both the dead load of the deck and the live load (specially when it is eccentric outwards) produce torsion in the deck thereby causing additional reaction over the normal reaction at outer edge or outer bearings at B and D but relief of some reaction at A and C. These aspets should be duly considered in the design of bearings, substructure and foundations.

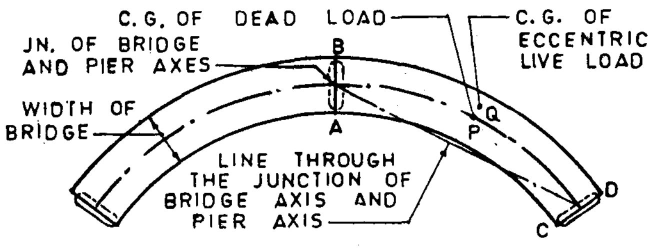


FIGURE 9.15

9.2.4.2 Another factor which induces additional reaction at B and D is the centrifugal force of the moving vehicles as outlined in Art. 5.10 in Chapter 5. The centrifugal force acting at a height of 1.2 m above the bridge deck will cause moment which is equal to the centrifugal force multiplied by the depth of deck or girder plus 1.2 m and this will induce additional reaction at B and D.

9.2.5 Design of Superstructure

9.2.5.1 As stated in Art. 9.2.4.1, both the dead load and the live load will induce torsion in the deck. This will not much affect the design of solid slab deck since the span is less and as such the torsional moment is less. However, the torsional stress may be checked and additional steel provided if the stress exceeds the permissible value. In addition, the inner corners A and C (where warping might take place due to deflection of the deck) shall be provided with some top reinforcement as in acute angle corners of a skew bridge. In girder bridges, the torsion due to dead and live load will thrust more load on the outer girder and give relief to inner girder in addition to the normal distribution of load as illustrated in Chapter 6. The bending of the bridge deck in plan due to lateral centrifugal force has also to be duly considered.

9.2.5.2 The centrifugal force will also cause torsion of deck which may be taken as equal to the centrifugal force multiplied by the distance from the cg. of the deck to 1.2 m above the deck. This torsional moment will again thrust more load on the outer girder and give relief to the inner girder. Therefore, the outer girder for a curved bridge has to carry more load than the outer girder for a normal straight bridge.

9.2.5.3 To prevent the overturning of the moving vehicles due to centrifugal force, superelevation in the bridge deck as given by the following equation shall be provided.

$$\text{Superelevation, } e = \frac{V^2}{225R} \qquad (9.1)$$

Where e = Superelevation in metre per metre
V = Speed in Km. per hour
R = Radius in metre.

Superelevation obtained from equation 9.1 shall be limited to 7 percent. On urban sections with frequent intersections it will, however, be desirable to limit the superelevation to 4 percent.

9.2.5.4 The superelevation may be provided in the deck slab by raising the deck slab towards the outer curve as shown in Fig. 9.16. The required superelevation may be achieved by increasing the height of the pedestals towards outer curve (keeping the depth of the girder same for all) as shown in Fig. 9.16a or by increasing depth of the girders towards outer curve (keeping the pedestal height same for all) as in Fig. 9.16b but the former is preferable to the latter from economic and constructional point of view.

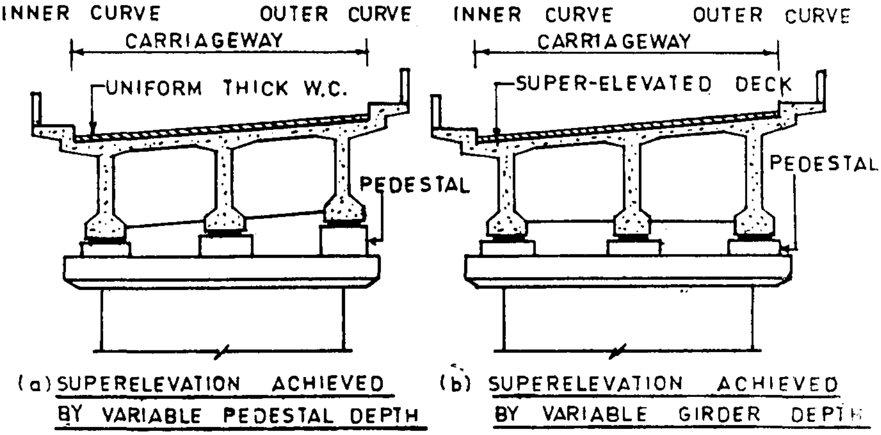


FIG. 9·16 METHOD OF PROVIDING SUPERELEVATION IN CURVED BRIDGES

9.2.6 Design of Bearings

9.2.6.1 In addition to the usual considerations for the design of the bearings, the effect of the centrifugal force and the torsional moment as outlined in Art. 9.2.4 shall be duly considered and the design of the bearings shall be made accordingly. The detailing of the bearings shall be such that the deck supported on the bearings is restrained from horizontal movement in the transverse direction due to the effect of centrifugal force in addition to the seismic force owing to dead and live loads.

9.2.7 Design of Substructure and Foundations

9.2.7.1 While preparing the design of substructure as well as the foundations, additional reaction on one side of the pier due to torsion as explained in Art. 9.2.4.1 and additional horizontal force at the top of the pier due to centrifugal force as mentioned in Art. 9.2.4.2 shall be given due consideration.

9.3 REFERENCES

1. Ministry of Shipping and Transport (Roads Wing), Standard Plans for Highway Bridges, Volume II — Concrete Slab Bridges (First Revision, 1983) — Indian roads Congress.
2. *"Concrete Bridges"* — The Concrete Association of India, Bombay 20.
3. Heins, C.P. and Firmage, D.A. — *"Design of Modern Steel Highway Bridges"* — A Willy Interscience Publication (John Wiley & Sons., Inc.), New York.
4. Rakshit, K.S. — *"The Layout of Bearings in Skew or Curved Bridges"* — Construction Engineers of India, Vol. IX — December, 1965. State Engineers' Association, Calcutta.

CHAPTER ***10***

REINFORCED CONCRETE CONTINUOUS BRIDGES

10.1 GENERAL

10.1.1 The types of superstructures dealt with in the previous chapters are all statically determinate structures. These structures have the relative advantage that their designs are simple and do not involve any complicated analysis but the main drawback is that such structures are generally comparatively costly.

10.1.2 Continuous bridges, on the otherhand, are more economical but the disadvantage of these types of bridges is their lack of simplicity in the design procedure. These structures are statically indeterminate and therefore, the structural analysis is very much laborious specially when it involves moving loads.

10.1.3 The advantages in favour of continuous bridges are :

i) Unlike simply supported bridges, these structures require only one line of bearings over piers thus reducing the number of bearings in the superstructure as well as the width of the piers.

ii) Due to reduction in the width of pier, less obstruction to flow and as such possibility of less scour.

iii) Require less number of expansion joints due to which both the initial cost and the maintenance cost become less. The riding quality over the bridge is thus improved.

iv) Reduces depth at midspan due to which vertical clearance or headroom is increased. This may bring down the bridge deck level reducing thereby not only the cost of the approaches but also the cost of substructure due to lesser height of piers and abutments which again reduces the cost of the foundation.

v) Better architectural appearance.

The disadvantages are :

i) Analysis is labourious and time consuming.

ii) Not suitable on yielding foundations. Differential settlement may cause undesirable stresses.

10.2 BEARINGS

10.2.1 In any unit of continuous structure, roller or sliding bearings over all the piers excepting one are provided. The one left out shall have hinged or rocker bearing for structural stability.

10.3 TYPE OF CONTINUOUS BRIDGES

10.3.1 Slab and T-beam Bridges

10.3.1.1 For sketch, Fig. 4.3 of Chapter 4 may be referred to. Solid slab continuous bridges may be adopted for spans upto 25 m, T-beam continuous bridges may be used for spans beyond 20 m. but below 40 m. Above this limit box girder bridges may be found suitable.

10.3.2 Box-girder Bridges

10.3.2.1 Box girder superstructures which are generally found useful for medium long span bridges consist of longitudinal girders usually three in number with deck and soffit slabs at top and bottom although single cell box girders is not uncommon. As the name implies, the longitudinal girders and the cross girders along with top and bottom slab form the box. The advantage of this type of superstructure is its great torsional resistance which helps a good deal in better distribution of eccentric live loads over the girders. Unlike girder bridges, live load distribution becomes more even in box girder bridges.

10.3.2.2 Another advantage that may be achieved from this type of structure is that instead of increasing the depth of the section where the resisting moment becomes less than the design moment, the former can be increased if the slab thickness on the compression side is suitably increased. To cater for varying moments at different sections, the thickness of the top or bottom slab is varied depending on whether positive or negative moment is to be resisted.

10.3.2.3 The deck slab is designed as a continuous slab over the longitudinal girders similar to slab and girder bridges. The thickness of deck slab varies from 200 to 250 mm. depending on the spacing of the longitudinal girders. The soffit slab thickness varies from 125 to 150 mm. where it has no structural function except forming the box but to resist negative moment it may be necessary to increase it upto 300 mm. near the support. The web thickness of the longitudinal girders is gradually increased towards the supports where the shear stresses are usually critical. Web thickness of nearly 200 mm. at the centre varying to 300 mm. at the support is normally found adequate. The web at the support is widened suitably to accommodate the bearings, the widening being gradual with a slope of 1 in 4.

10.3.2.4 The diaphragms are provided in the box girder to make it more rigid as well as to assist in even distribution of live load between the girders. For better functioning, their spacing should be between 6 m. to 8 m. depending on the span lengths. It is advisable to provide at least 5 diaphragms in each span — two at supports, two at quarter span and one at the midspan. Openings are kept in the diaphragms to facilitate removal of shutterings from inside the boxes (Fig. 11.5). Suitable manholes may be kept in the soffit slab for this purpose also. These may be covered by manhole covers of precast concrete.

10.3.2.5 About 40 percent of the main longitudinal tensile reinforcement are distributed over the tension flange uniformly, the remaining 60 percent being concentrated in the webs in more than one layer if necessary. In deep girder bridges, a considerable depth of the web below the top flange near the support is subjected to tensile stress. To cater for this tensile stress it is recommended that about 10 percent of the longitudinal reinforcement may be provided in this zone unless inclined stirrups are used for diagonal tension.

10.4 PROPORTIONING OF STRUCTURES

10.4.1 Equal spans are sometimes adopted for various reasons one of them being architectural consideration but for economical design, the intermediate spans should be relatively more in length than the end spans. Generally, the following ratios of intermediate to end span are found satisfactory —

a)	Slab bridges —	
	i) End span less than 10 m.	1.26
	ii) End span 10 m. and more	1.31
b)	Girder bridges —	
	End span 20 m. and above	1.37
c)	Box Bridges —	
	End span 30 m. and above	1.40

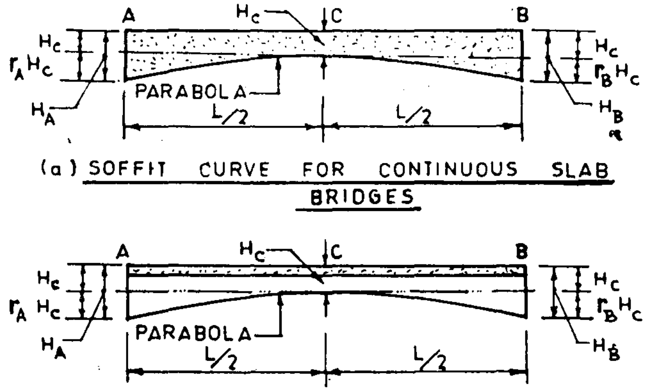
10.4.2 In a continuous bridge, the moment of inertia should follow the moment requirement for a balanced and economical design. This is achieved by making the bottom profile parabolic as shown in Fig. 10.1. Sometimes, straight haunches or segmental curves are provided near supports to get the increased depth required from moment consideration.

The soffit curves shown in Fig. 10.1 are made up of two parabolas having the apex at the centre line of the span. For symmetrical soffit curves,

$$r_A = r_B = r \text{ (say)}$$

where "r" is the ratio of increase in depth at supports to the depth at the centre line of span.

The following values of "r" have been recommended for slab bridges —



(a) SOFFIT CURVE FOR CONTINUOUS SLAB BRIDGES

(b) SOFFIT CURVE FOR CONTINUOUS T-GIRDER BRIDGES

FIG. 10·1 TYPICAL SOFFIT CURVES (CAI)[1]

a) End span 10 m or less,
$r = 0$ for all spans

b) End span between 10 m and 15 m,
i) $r = 0$ to 0.4 for outer end span
ii) $r = 0.4$ at first interior support
iii) $r = 0.5$ at all other supports

The values of r_A and r_B for girder bridges may be computed from the following formulae :

$$r_A = \sqrt[3]{\frac{I_A}{I_C}} - 1 \quad \text{and} \quad r_B = \sqrt[3]{\frac{I_B}{I_C}} - 1$$

Where I_A, I_B and I_C are the moment of inertia of the T-beam at A, B and mid-span respectively.

For girder bridges, the undermentioned values of "r" have been recommended —

i) Outer end of end spans, $r = 0$
ii) 3 span unit, $r = 1.3$ at intermediate supports.
iii) 4 span units, $r = 1.5$ at centre support and 1.3 at the first interior support.

10.5 METHOD OF ANALYSIS

10.5.1 Continuous structures may be analysed by various methods but most common method is the moment distribution. When haunches are used, the analysis becomes more complicated and therefore, design tables and curves have been made available for structures with various types of haunches such as straight, segmental, parabolic etc. as well as for various values of r_A, r_B etc. One such reference literature is *"The Applications of Moment Distribution"* published by the Concrete Association of India, Bombay. These tables and curves give the values of fixed end moments, carryover factors, stiffness factors etc. from which the nett moments on the members after final distribution may be worked out.

10.6 INFLUENCE LINES

10.6.1 Fig. 10.2 shows some influence line diagrams at different sections for a three equal span continuous bridge having constant moment of inertia. To get reaction or moment at a point due to a concentrated load, W, the ordinate of the appropriate influence line diagram is to be multiplied by W. For uniformly distributed load w, reaction or moment = (Area of appropriate influence line diag) × w.

The influence line diagrams for moments, shears, reactions etc. for continuous structure with variable moment of inertia may be drawn in a similar way, the ordinates for the influence line diagrams being determined taking into consideration the appropriate frame constants for the given structures.

The design live load moments, shears and reactions at different sections are calculated by placing the live loads on the appropriate influence line diagrams. The loads should be placed in such manner that maximum effect is produced in the section under consideration.

10.7 DESIGN PROCEDURE

1. Fix up span lengths in the unit and select rough sections at midspans and at supports.
2. Select appropriate soffit curve.
3. Work out dead load moments at different sections. This may be done as follows :
   i) Find the fixed end moments.
   ii) Find the distribution factors and carryover factors for the unit.

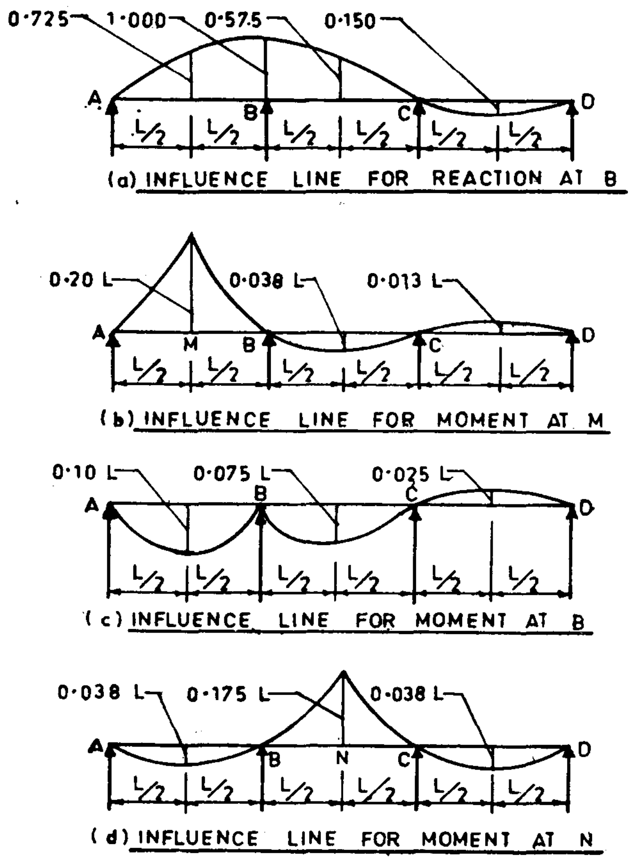


FIGURE 10·2 (Reynolds)[2]

iii) Distribute the fixed end moments by Moment Distribution Method. This will give the elastic moments. Add to it the free moment due to dead load.

4. Draw influence line diagrams for moments (vide Art 5.20). The procedure is as follows :
   i) Find the F.E.M. for unit load on any position.
   ii) Distribute the F.E.M. and find out the elastic moments after correction for sway where necessary.
   iii) Add free moment to elastic moment. The moments so obtained at a particulars section for various load positions will give the ordinates of the BM influence line diagram at the locations on which unit load is placed.
   iv) Repeat process (i) to (iii) above and get the ordinates of the influence line diagram for various sections.
5. Work out live load moments at different sections.
6. Combine the live load moments with the dead load moments so as to get the maximum effect.
7. Check the concrete stress and calculate the area of reinforcement required.

8. Draw influence line diagrams for shears as before for various sections. Estimate both the dead load and live load shear and check the shear stress at the critical sections and provide necessary shear reinforcement where necessary.
9. Detail out the reinforcement in the members such that all the sections are adequately catered for respective critical bending moments and shear forces.

10.8 REFERENCES

1. *"Concrete Bridges"* — The Concrete Association of India, Bombay 20.
2. Reynolds, C.E. — *"Reinforced Concrete Designers' Handbook (Fifth Edition)"* — Concrete Publications Ltd., 14, Dartmouth Street, London, S.W.1.

Chapter 11

REINFORCED CONCRETE BALANCED CANTILEVER BRIDGES

11.1 GENERAL

11.1.1 Balanced cantilever bridges are adopted for comparatively longer spans where simply supported, continuous or rigid frame type superstructures are found unsuitable. Simply supported decks of any type having spans more than 20 to 25 m. require comparatively greater depths and therefore, become uneconomical. On the otherhand, continuous or rigid frame type bridges, though cheaper, must be founded on unyielding foundations since otherwise unequal settlement of the foundations may induce harmful stresses and thereby cracks may develop in the members. Balanced cantilever bridges are combination of the simply supported and continuous structures. They have the advantages of simply supported as well as continuous structures. viz. (1) the structures are statically determinate and the moments, shears etc., may be found out by the basic rules of statics and (2) the possibility of cracks due to unequal settlement of the foundations is eliminated. (3) This type of structure is also comparable to some extent with continuous structures since the free positive moment at midspan is partly balanced by the negative moment caused by the cantilever and thereby leads to economy

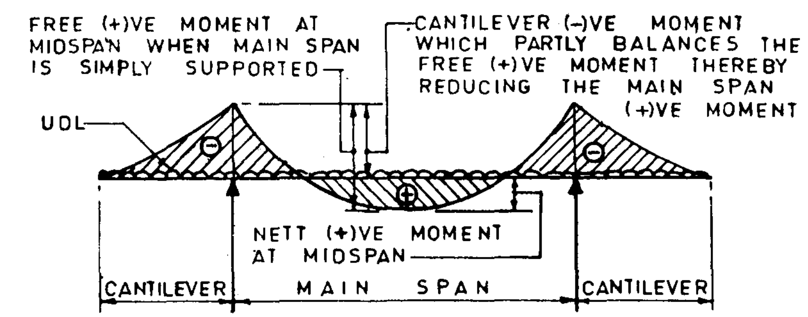


FIG. 11·1 MOMENTS IN BALANCED CANTILEVER BRIDGE DUE TO UDL in materials. (4) Balanced cantilever bridges also require one line of bearings over the piers similar to continuous bridges.

11.1.2 For bridging smaller channels, usually one central longer span with two shorter end spans of the types as shown in Fig. 4.4a and 4.4b are adopted but where the bridge length is more, repetition of the type of span illustrated in Fig. 11.2 is resorted to.

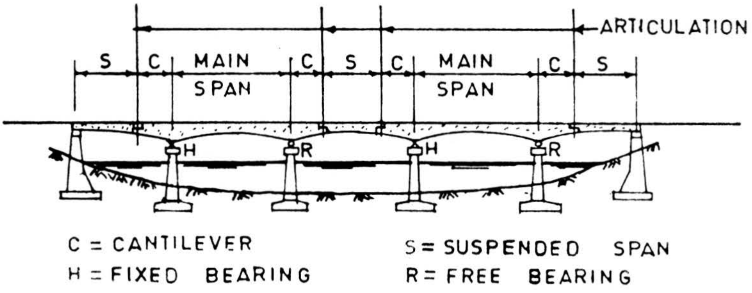


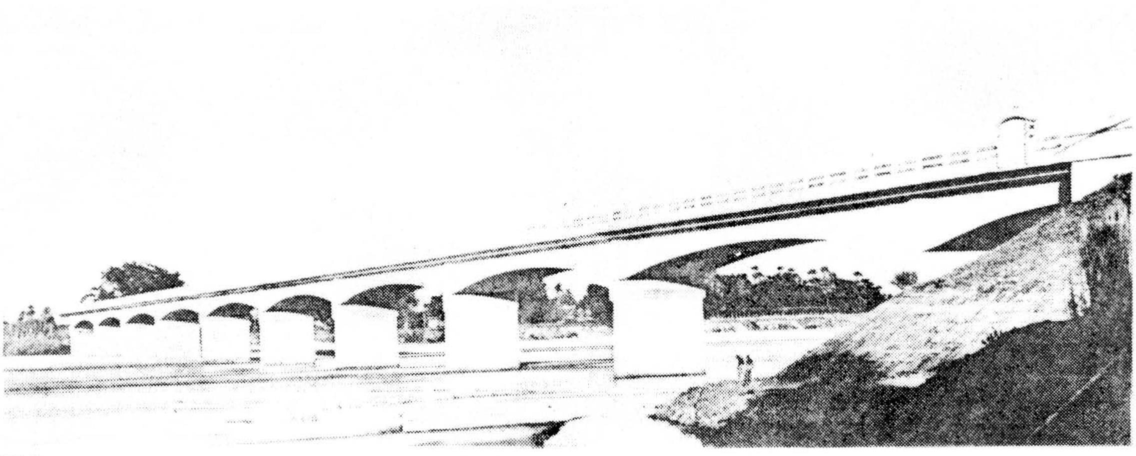
FIG. 11·2 MULTISPAN BALANCED CANTILEVER BRIDGE

11.2 TYPES OF SUPERSTRUCTURE

11.2.1 The superstructures may be of solid slab, T-beam and slab, hollow box girder etc. For details, Chapter 4 may be referred to. Photograph 3 shows one hollow-box balanced cantilever bridge.

11.3 PROPORTIONING OF MEMBERS

11.3.1 To get the most economical design, the proportioning of the members should be such that the sections at midspan and at support satisfy both the structural and architectural requirements and at the same time require



Photograph 3

BRIDGE OVER DWARAKESWAR RIVER
(R. C. Hollow-box Girder Bridge on Bishnupur — Sonamukhi Road in the District of Bankura, West Bengal — consists of 9 (nine) intermediate spans of 33.0 m and 2 (two) end spans off 15.0 m having total length of 327.0 m). *(Highway Bridges)*[3]

minimum quantity of materials. To achieve this, the cantilever lengths are usually made from 0.20 to 0.30 of the main span. This ratio depends on the length of the main span and the type of suspended span the cantilever has to support as well as the number of cantilevers (single or double) available for balancing the midspan positive moment etc. For structures with only one cantilever, the cantilever lengths should be made relatively small otherwise there may be possibility of uplift at the other end.

11.3.2 The Author had studied the economics of solid slab balanced cantilever bridges in great details and shown that for economical design of solid slab balanced cantilever bridges with double cantilevers (i.e., for multi-span bridges), the ratio of cantilever to main span lies between 0.30 to 0.35 for decks having parabolic soffit with variable depth and 0.175 for decks with uniform depth (vide Reference 7).

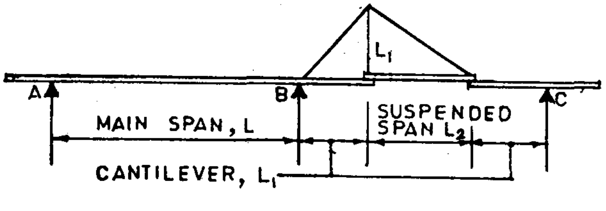
11.3.3 It has been observed that the moment at support is greater than that at midspan and therefore, the depth required at support is more than the same at midspan. The additional depth at support is achieved by providing haunches either straight or segmental near the supports. Sometimes the full span length is covered by the parabolic soffit profile as shown in Fig. 11.2. In such cases, though the depth at midspan required from design considerations should be more than at the ends of the suspended span, or near the quarter span, the same parabolic soffit profile is maintained from architectural considerations. Parabolic soffit profile is generally preferred to straight or segmental haunches from aesthetic point of view.

11.3.4 In order to meet the design requirements, the depth at midspan shall be between one-twentieth to one-thirtieth of the span length. The depth at support is normally 2 to 3 times the depth at mid span.

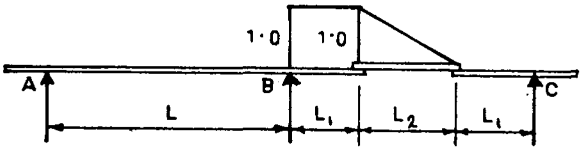
11.4 DESIGN CONSIDERATIONS

11.4.1 The suspended span is a simply supported structure and therefore, may be designed as such which has already been dealt with in Chapters 7 and 8.

11.4.2 The moments and shears for the cantilever arms are to be determined with loads on the cantilever alone or on the cantilever and the suspended span. The influence line diagrams for moment and shear for cantilever section near support are indicated in Fig. 11.3 from which the loading position for maximum moment



(a) INFLUENCE LINE DIAG. FOR MOMENT AT B



(b) INFLUENCE LINE DIAG. FOR SHEAR AT RIGHT OF SUPPORT B

FIG. 11·3 INFLUENCE LINE DIAGRAMS AT SUPPORT or shear may be found out. In designing the cantilever sections, both the dead and the live load moments or the shears are to be added together so as to get the design moments and shears.

11.4.3 It is interesting to note from the influence line diagrams for cantilever arm that the load on the main span has no effect either on the moment or on the shear of the cantilever section.

11.4.4 While both the dead and live load moments and shears are additive in designing the cantilever sections, the design of the main span sections, however, needs careful examination in arriving at the design moments and shears. At some sections of the main span near mid span, the live load moment may be of opposite nature to the dead load moments. In such cases it is not enough to design only for the combined dead and live load moments for the fact that the sections may not be safe to cater for the extra live load moment that is caused due to any possible overloading and as such there may not remain any factor of safety at these sections which is otherwise kept at all other parts of of the structure. Hence, the rule is that for sections where the dead and live load moments may be of opposite sign, the dead load moment must be divided by the factor of safety say 2 before adding it to the live load moment. This statement is further clarified in the following paragraph.

11.4.5 Let the dead load and live load moment at mid span section be (+) 1200 KNm and (-) 700 KNm, respectively. The nett design moment is therefore, (+) 500 KNm which is less than the DLM of (+) 1200 KNm for which the section is checked and reinforcement provided at the bottom of the section for +ve moment. Now if the live load moment is increased by 100 percent due to unusual conditions, the design moment for the abnormal condition will be $(+1200 - 1400) = (-)\,200$ KNm but the section has not been checked for this moment and moreover no steel at top of section to cater for the negative moment has been provided making thereby the section having no reinforcement against possible overloading. On the otherhand, if the dead load moment is reduced by a factor of safety 2 as mentioned in Art. 11.4.4, the design moment becomes $(+)\,\frac{1200}{2} - 700 = (-)\,100$ KNm and as such the section is capable of resisting a moment of (-) 200 KNm in case of possible overloading since the allowable stresses also may be doubled in such case to reach the ultimate strength of the reinforcement provided for resisting a moment of (–) 100 KNm.

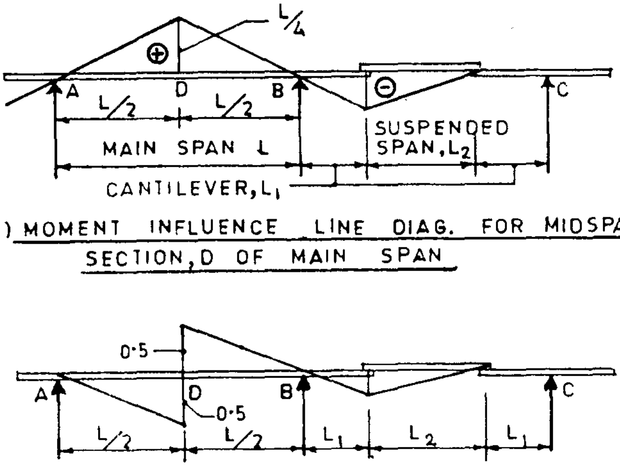
11.4.6 It is needless to mention that the reversal of nature of moments near the mid span section may occur in continuous structures also and proper care should be taken against this possibilities.

11.4.7 The influence line diagrams for moment and shear for the mid section of main span are illustrated in Fig. 11.4. The maximum +ve and -ve live load moments and shears may be evaluated by placing the live loads suitably on the influence line diagrams for getting maximum values.

11.4.8 In calculating shear forces at different sections, it is necessary to account for the correction due to haunches. The haunch correction necessary for this purpose may be given by the following equation :

$$V' = V \pm \frac{M}{d} \tan\beta \qquad (11.1)$$

Where
- V' = Corrected shear
- V = Un-corrected shear
- M = Bending moment at section under consideration due to loads corresponding to shear V
- d = Effective depth
- β = The angle between the top and bottom edges of the beam at that section.



(a) MOMENT INFLUENCE LINE DIAG. FOR MIDSPAN SECTION, D OF MAIN SPAN

(b) SHEAR INFLUENCE LINE DIAG. FOR MIDSPAN SECTION, D OF MAIN SPAN

FIG.11·4 INFLUENCE LINE DIAGRAMS AT MIDSPAN OF MAIN SPAN

The positive sign applies where the bending moment decreases with the increase in "d" (e.g. the haunches of simply supported beams). The negative sign applies when the bending moment increases with increase in "d" (as at haunches near the interior supports of continuous or balanced cantilever structures).

11.5 DESIGN PROCEDURE

1. Decide span lengths and assume rough sections of the main girders at important sections such as end support, intermediate support, mid span etc.
2. Select suitable profile of the soffit of the girders and find the depths at different sections of the girders.
3. Assume sections of cross girder and thickness of the deck and soffit slab.
4. Calculate the dead load bending moment at various sections.
5. Draw influence line diagram for moments for various sections.
6. Work out live load moments at different sections.
7. Check the adequacy of the sections in respect of concrete stresses and calculate the tensile reinforcement from the design moments which are obtained by combining the dead load moments with the live load moments, where necessary, in order to get maximum values for the entire deck.
8. Similar to moments, find the dead load and live load shears at different sections and check concrete stresses. If necessary, provide shear reinforcement.
9. Arrange the reinforcement properly so as to get the maximum out-turn from them.

ILLUSTRATIVE EXAMPLE 11.1

A hollow box balanced cantilever girder bridge with 7.5 m. roadway and 1.5 m. footpath on either side having spans as shown in Fig. 11.5 is to be designed for single lane of IRC Class 70-R or 2 lanes of IRC Class A

loading. Give brief outlines for calculating the bending moments and shear forces and draw the bending moment and shear force diagrams.

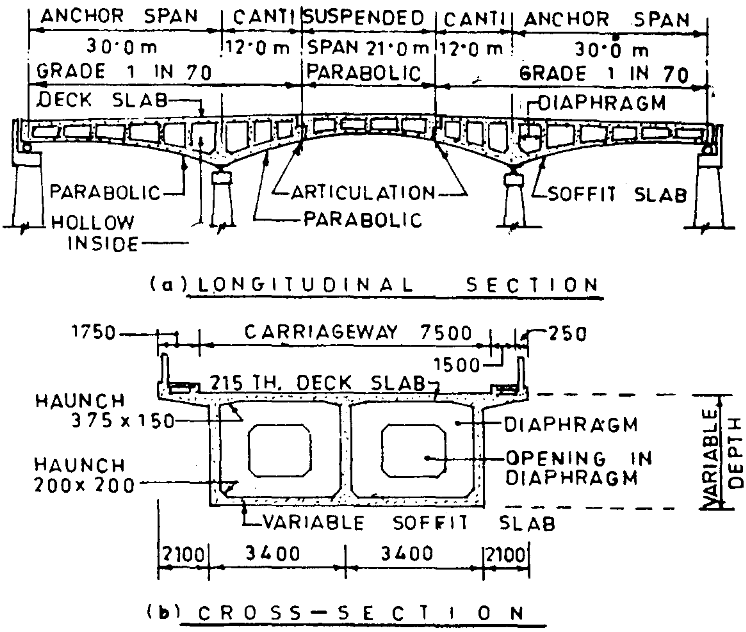


FIG.11·5 DETAILS OF HOLLOW-BOX GIRDER (Ex. 11·1)

Solution

The depths of the main girders over abutments and pier are assumed tentatively as shown in Fig. 11.6. The depths at other sections may be known if the variation of the top and bottom profiles are known.

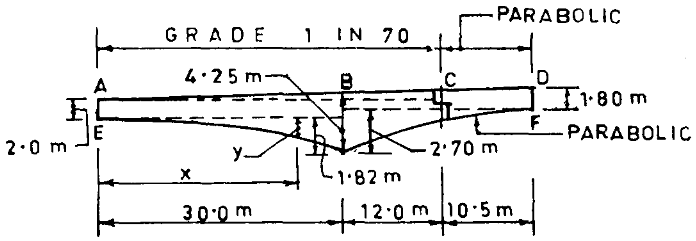


FIG. 11·6 TOP AND BOTTOM PROFILE OF MAIN GIRDER (Example 11·1)

Top Profile

a) *Anchor span with cantilever*

Straight line profile with grade of 1 in 70. Equation of the profile is given by,

$$y = m\,x = \frac{x}{70}$$

$$\text{i.e. } y = 0.0143\,x \text{ (origin at A)} \tag{11.2}$$

b) *Suspended span*

The shape of the top profile is parabolic. The equation of the parabola may be written in the form

$$y = kx^2 \tag{11.3}$$

The origin of the curve is at D and k is a constant the value of which may be determined in the following manner.

$$\text{Differentiating equation 11.3, } \frac{dy}{dx} = 2kx \tag{11.4}$$

At C, x = 10.5 m. and slope, $\frac{dy}{dx} = \frac{1}{70}$

From equation 11.4, $k = \frac{1}{70 \times 2 \times 10.5} = 0.00068$

Hence equation 11.3 comes to $y = 0.00068\,x^2$ (origin at D)

$\therefore$ Fall of C from D = $0.00068\ (10.5)^2 = 0.075$ m.

Fall of B from C = $\frac{12.0}{70} = 0.17$ m.; Fall of A from B = $\frac{30.0}{70} = 0.43$ m.

Bottom Profile

a) *Anchor span*

Equation of the parabola, $y = kx^2$

When x = 30.0 m, y = 1.82 m. $\therefore k = \frac{y}{x^2} = \frac{1.82}{(30)^2} = 0.002$

$\therefore$ The equation of the bottom profile becomes, $y = 0.002\,x^2$ ... (origin at E)

b) *Cantilever and the suspended span*

Equation of the parabola, $y = kx^2$, when x = 22.5 m, y = 2.70 m. $\therefore k = \frac{y}{x^2} = \frac{2.70}{(22.5)^2} = 0.00533$

$\therefore$ The equation becomes, $y = 0.00533\,x^2$ ... (origin at F)

The depths at various sections may be found out from the above equations, for example, the depth at the mid section of anchor span may be given by $D = 2.0 + y_1 + y_2$

$$= 2.0 + 0.0143x + 0.002\,x^2$$

$$= 2.0 + 0.0143 \times 15.0 + 0.002\,(15.0)^2$$

$$= 2.0 + 0.2145 + 0.45 = 2.6645 \text{ m.}$$

Dead Load Calculation

The udl due to deck slab, soffit slab, wearing course, wheel guard, railings and railing posts etc., is calculated as in Chapter 8. The weight of the longitudinal beams may be assumed to act as udl between two sections (say

3m apart) the udl being calculated with average depth and thickness of the rib between the sections under consideration. The cross beam or diaphragm load shall be taken as concentrated load. These loads are shown in Fig. 11.7.

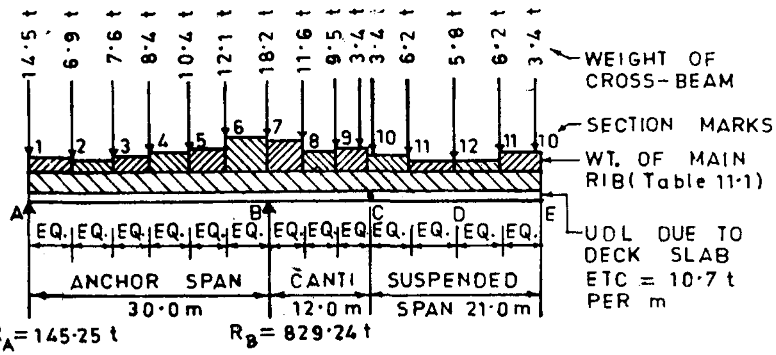


FIG. 11·7 DEAD LOAD OF SUPERSTRUCTURE (Ex. 11·1)

TABLE 11.1 : DEAD LOAD OF RIBS OF LONGITUDINAL GIRDER PER METRE

UDL due to	UDL in tonnes between section										
	1&2	2&3	3&4	4&5	5&6	6&7	7&8	8&9	9&10	10&11	11&12
Longitudinal rib	4.03	3.05	3.48	4.56	6.00	11.64	11.35	5.51	6.49	5.25	2.49

The dead load moments at various sections are computed with the loads shown in Fig. 11.7 and the values shown in table 11.2. The moments for the anchor span and the cantilever are worked out for two conditions viz.

Case I. Working condition with the suspended span over the cantilever arm.

Case II. Condition during construction period without the suspended span. This case may also occur if due to any reason the suspended span is dislodged from its place during its service period. Under this condition no live load will act on the bridge.

Live Load Moments

The live load moments (both positive and negative) at various sections may be worked out by placing the live loads on the respective influence line diagrams. Appropriate impact allowance should also be made in the evaluation of the live load moments. To these values, the moments due to footway loading should also be added. The design moments are obtained by adding both the dead and live load moments including those due to the footway loading.

The evaluation of live load moment at the centre of anchor span is shown below as an illustration. The moments for other sections are to be calculated in the similar way.

For maximum positive and negative moment at mid section of anchor span, the position of single lane of Class A load will be as shown in Fig. 11.8. Class 70-R load will not produce worse effect. For distance between loads, refer to Fig. 5.2.

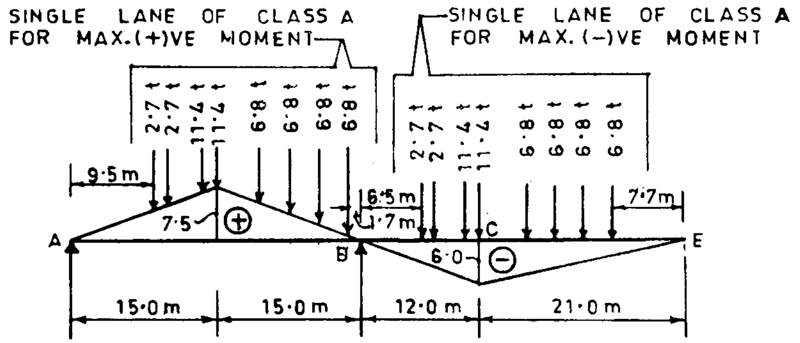


FIG. 11·8 LOAD POSITION ON THE INFLUENCE LINE DIAGRAM FOR MAXIMUM MOMENT AT CENTRE OF ANCHOR SPAN (Example 11·1)

Maximum positive moment

$$= \frac{7.5}{15.0} [\, 2.7\,(9.5 + 10.6) + 11.4\,(13.8 + 15.0) + 6.8\,(10.7 + 7.7 + 4.7 + 1.7)\,]$$

$$= \tfrac{1}{2}\,(54.27 + 328.32 + 168.64) = 275.62 \text{ tm}$$

From Art. 5.6.2 and 5.6.6

Impactor factor $= \dfrac{4.5}{6 + L} = \dfrac{4.5}{6 + 30} = 0.125$

Maximum positive moment with impact for single lane load = $1.125 \times 275.62 = 310.07$ tm

For 2 lanes of Class A load,

Max. positive moment = $2 \times 310.07 = 620.14$ tm. say 620.2 tm.

Similarly from Art. 5.6.2 and 5.6.6, impact factor for load on cantilever

$$= \frac{4.5}{6 + L} = \frac{4.5}{6 + (12.0 + \frac{1}{2} \times 21.0)} = \frac{4.5}{(6 + 22.5)} = 0.158$$

Impact factor for load on suspended span $= \dfrac{4.5}{(6 + 21.0)} = 0.166$

∴ Maximum negative moment with impact (Fig. 11.8) for single lane of Class A loading

$$= 1.158 \times \frac{6.0}{12.0} [\, 2.7\,(6.5 + 7.6) + 11.4\,(10.8 + 12.0)\,] + 1.166 \times \frac{12.0}{21.0} [\, 6.8\,(16.7 + 13.7 + 10.7 + 7.7)\,]$$

$$= 0.579\,(38.07 + 259.92) + 0.333\,(331.84) \quad = 172.54 + 110.50 = 283.04 \text{ tm}$$

For 2 lanes of Class A loading = $2 \times 283.04 = 566.08$ tm. Say 566.10 tm.

Footway Loading (Vide Chapter 5)

a) *For Anchor Span*

Intensity of loading $= P' - \dfrac{(40L - 300)}{9} = 400 - \dfrac{(40 \times 30 - 300)}{9} = 400 - 100 = 300$ Kg/m^2

Footway loading per metre of bridge = $2 \times 1.5 \times 300 = 900$ Kg.

b) *For Cantilever Span*

Intensity of loading $= P' - \dfrac{(40L - 300)}{9} = 400 - \dfrac{(40 \times 12.0 - 300)}{9} = 400 - 20 = 380 \text{ Kg/m}^2$

Footway loading per metre of bridge = $2 \times 1.5 \times 380 = 1140$ Kg.

c) *For suspended Span*

Intensity of loading $= P' - \dfrac{(40L - 300)}{9} = 400 - \dfrac{(40 \times 21 - 300)}{9} = 400 - 60 = 340 \text{ Kg/m}^2$

$\therefore$ Footway loading per metre of bridge = $2 \times 1.5 \times 340 = 1020$ Kg.

In calculating the positive moment at mid section of anchor span due to footway loading, only the anchor span will be assumed to be loaded with footway loading. On the otherhand, the cantilever and the suspended span will be loaded for negative moment at the section.

From influence line diag. (Fig. 11.8)

Positive moment = Area of influence line diagram × intensity of load
$= \frac{1}{2} \times 30.0 \times 7.5 \times 900 = 1{,}01{,}000$ Kgm = 101 tm

Negative moment $= \frac{1}{2} \times 12.0 \times 6.0 \times 1140 + \frac{1}{2} \times 21.0 \times 6.0 \times 1020.$

$= 41{,}000 + 64{,}000 = 1{,}05{,}000$ Kgm = 105 tm

Total positive live load moment = 620.2 + 101 = 721.2 tm

Total negative live load moment = 566.1 + 105 = 671.1 tm

TABLE 11.2 : DESIGN MOMENTS IN TONNES-METRE

Location	Section	Case - I Service Condition			Case - II Construction or failure condition (dead load only)	Design moment
		Dead load moment	Live load moment	Dead plus live load moment		
Abutment	1	—	—	—	—	—
Anchor span	2	502.6	432.0 -216.0	934.6	849.6	934.6
	3	586.1	676.8 -446.4	1262.9 -153.4(*)	1281.6	1281.6 -153.4
	4	256.3	721.2 -671.1	977.5 -543.0(*)	1300.3	1300.3 -543.0
	5	-535.7	679.7 -894.2	411.85(*) -1429.9	881.3	881.3 -1429.9
	6	-1,792.8	433.4 -1117.4	-2910.2	-21.6	-2910.2

TABLE 11.2 : DESIGN MOMENTS IN TONNES-METRE *Contd.*

Location	Section	Case - I Service Condition			Case - II Construction or failure condition (dead load only)	Design moment
		Dead load moment	Live load moment	Dead plus live load moment		
Pier	7	-3636.0	-1340.6	-4976.6	-1513.4	-4976.6
Cantilever	8	-2066.4	-849.6	-2916.0	-648.0	-2916.0
	9	-868.3	-380.2	-1248.5	-168.5	-1248.5
Articulation	10	—	—	—	—	—
Suspended span	11	676.8	354.2	1031.0	—	1031.0
	12	888.5	434.9	1324.4	—	1324.4

(*) As explained earlier in this Chapter, the dead load moments are to be divided by 2 before adding these to the live load moments where the live load moments are of opposite nature to those due to dead loads.

Thus in Section 3,

Design moment $= \frac{586.1}{2} - 446.4 = (-)153.4$ tm

In Section 4, design moment $= \frac{256.3}{2} - 671.1 = (-)543.0$ tm

In Section 5, design moment $= (-)\frac{535.7}{2} + 679.7 = 411.85$ tm

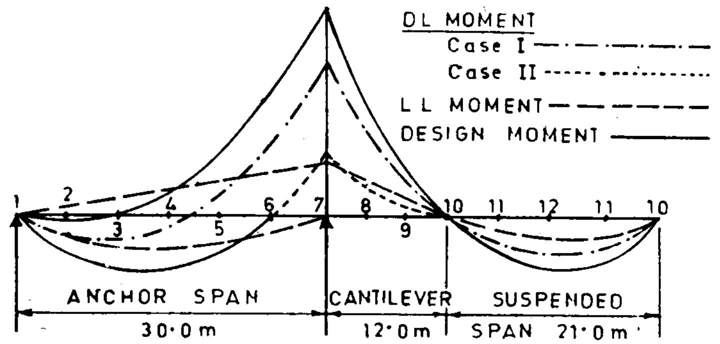


FIG. 11·9 BENDING MOMENT DIAGRAMS FOR ANCHOR SPAN, CANTILEVER AND SUSPENDED SPAN

Dead Load Shear

Sign convention —

Upwards to the left and downwards to the right of section = +ve shear and vice versa.

The dead load shear forces at different sections are calculated with the loads and reactions shown in Fig. 11.7.

As stated previously in this chapter, the top and bottom of the girders are provided with curved profiles and therefore, haunch correction is necessary. The shears obtained above are uncorrected shears and hence are to be corrected. The method of shear calculation is illustrated below for Section 2 (left).

Uncorrected shear at Section 2 (left) = $145.25 - 14.5 - (10.7 + 4.03) \times 5.0 = 57.1$ t

Corrected shear is given by Equation 11.1 which is

$$V' = V \pm \frac{M}{d} \tan \beta, \qquad M = 502.6 \text{ tm}, d = 2.05 \text{ m}.$$

$$\tan \beta_1 = \frac{1}{70} = 0.0143 \quad \therefore \beta_1 = 0° - 49'\text{–}0''$$

$$\tan \beta_2 = \frac{dy}{dx} = 2kx = 2 \times 0.002 \times 16.67 = 0.0667 \quad \therefore \beta_2 = 1° - 10'\text{–}0''$$

$$\text{or } \tan \beta = \tan (\beta_1 + \beta_2) = \tan (0° - 49'\text{-}0'' + 1° - 10'\text{-}0'') = \tan 1° - 59'\text{-}0'' = 0.0347$$

$$\therefore V' = 57.1 - \frac{502.6}{2.05} \times 0.0347 = 48.59 \text{ t}$$

Live Load Shear

The live load shear at any section can be evaluated by placing appropriate live loads on the shear influence line diagram. Since haunch correction in the live load shear values is necessary due to the presence of the top and bottom curved profiles, it is desirable that the shear influence line diagram is corrected for the above. In this process, M of the expression $\frac{M}{d} \tan \beta$ is the live load moment at the section for the unit load at that location at which the ordinate for shear influence line diagram is to be drawn.

As before, let us find out the live load corrected shear at Section 2 (left).

Influence line ordinate (uncorrected) Section 2 (left) = 0.8333.

$$M = \frac{ab}{L} = \frac{5.0 \times 25.0}{30.0} = 4.17 \text{ tm}.$$

$$\therefore \text{Corrected ordinate, } V' = V - \frac{M}{d} \tan \beta = 0.8333 - \frac{4.17}{2.05} \times 0.0347 = 0.7627$$

2 lanes of Class A load will produce maximum shear.

Maximum positive live load shear for single lane loading (Fig. 11.10)

$$= \frac{0.7627}{25.0} [\ 11.4\ (25.0 + 23.8) + 6.8\ (19.5 + 16.5 + 13.5 + 10.5)\ ] - \frac{0.2373}{5.0} [\ 2.7\ (0.7 + 1.8)\ ]$$

$$= 0.0305\ (556.32 + 408) - 0.0476 \times 6.75 = 29.41 - 0.32 = 29.09 \text{ t}$$

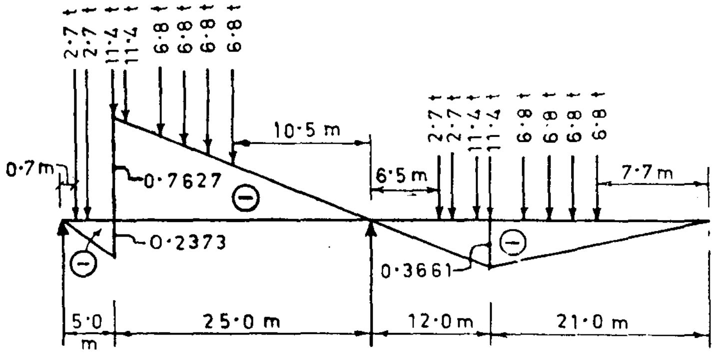


FIG.11·10 LIVE LOAD SHEAR AT SEC. 2 (Ex. 11·1)

For 2 lanes of Class A loading and with 12.5 percent impact, maximum live load shear
$= 2 \times 1.125 \times 29.09 = 65.45$ t.

Maximum negative live load shear with impact for single lane loading

$$= 1.166 \times \frac{0.3661}{21.0} [\, 6.8\,(16.7 + 13.7 + 10.7 + 7.7)\,] + 1.158 \times \frac{0.3661}{12.0} [\, 11.4\,(12.0 + 10.8) + 2.7\,(7.6 + 6.5)\,]$$

$= 0.0203\,(331.84) + 0.0353\,(259.92 + 38.07)$

$= 6.736 + 10.519 = 17.255$ t

For 2 lanes of Class A loading, max. -ve L.L. shear $= 2 \times 17.255 = 34.51$ t

Live load shear due to footway loading —

From influence line diagram (Fig. 11.10),

Positive L.L. shear = Area of influence line diagram × intensity of loading
$= \frac{1}{2} \times 0.7627 \times 25.0 \times 900 - \frac{1}{2} \times 0.2373 \times 5.0 \times 900 = 8580 - 534 = 8046$ Kg. = 8.05 t.

Negative L.L. shear
$= \frac{1}{2} \times 0.3661 \times 12.0 \times 1140 + \frac{1}{2} \times 0.3661 \times 21.0 \times 1020 = 2504 + 3921 = 6425$ Kg. = 6.43 t.

∴ Total L.L. positive shear = 65.45 + 8.05 = 73.5 t.

Total L.L. negative shear = 34.51 + 6.43 = 40.94 t.

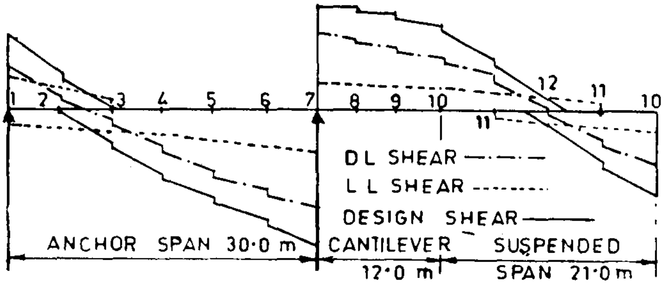


FIG. 11·11 SHEAR FORCE DIAG. FOR LONGITUDINAL RIBS

The live load shears for other sections also may be obtained in the above manner. The typical nature of shear force diagram for dead load, live load etc. is shown in Fig. 11.11.

11.6 DESIGN OF ARTICULATION

11.6.1 The articulation of a cantilever bridge is the most vulnerable part in the structure and therefore, special attention should be paid to both the design and construction of this important component.

11.6.2 The articulation is subjected to the following forces :

i) Vertical reaction "R" from the suspended span due to dead and live load reactions including the changes in the reaction due to braking, wind or seismic forces.

ii) Horizontal force "H" due to braking, seismic, temperature etc.

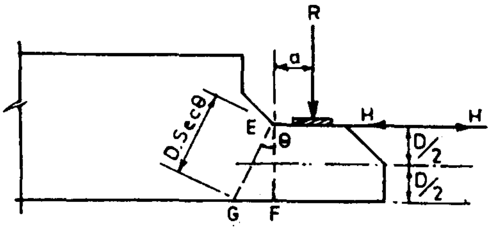


FIG. 11·12 LOADS AND FORCES ON ARTICULATION

The combined effect of the above forces makes the plane of maximum bending stress inclined at an angle θ with the vertical instead of being parallel to it.

11.6.3 The design of the articulation should cater for the following :

i) Sufficient tensile steel is to be provided to resist both the bending and the direct tensile stress at the inclined plane (i.e. plane of maximum stress).

ii) The vertical plane at the neck should also be properly reinforced to cater for the tensile stress due to both bending and direct stress.

iii) Necessary shear reinforcement at both the vertical plane and the inclined plane (i.e. the plane of maximum shear) are to be provided.

Assuming "B" as width of the articulation, and referring to Fig. 11.12,

Cross-sectional area of inclined plane EG = A = BD sec θ.

$$\text{Direct tension on the plane, } P = (R \sin\theta + H \cos\theta) \qquad (11.5)$$

$$\therefore \text{ Direct tensile stress } = \frac{P}{A} = \frac{(R \sin\theta + H\cos\theta)}{BD \sec\theta}$$

$$\text{Section modulus of the plane } = Z = \frac{BD^2 \operatorname{Sec}^2\theta}{6}$$

$$\text{Bending moment on the plane } = R\left(a + \frac{D}{2}\tan\theta\right) + H\frac{D}{2} \qquad (11.6)$$

$$\therefore \text{ Bending tensile stress, } \frac{M}{Z} = \frac{R\left(a + \dfrac{D}{2}\tan\theta\right) + \dfrac{HD}{2}}{\dfrac{BD^2 \operatorname{Sec}^2\theta}{6}}$$

$$\therefore \text{ Nett stress, } f = \frac{P}{A} + \frac{M}{Z}$$

$$= \frac{R \sin\theta + H\cos\theta}{BD\sec\theta} + \frac{R\left(a + \frac{D}{2}\tan\theta\right) + \frac{HD}{2}}{\frac{BD^2\,Sec^2\theta}{6}}$$

$$= \frac{\cos\theta}{BD}(R\sin\theta + H\cos\theta) + \frac{6\cos^2\theta}{BD^2}\left[R\left(a + \frac{D}{2}\tan\theta\right) + \frac{HD}{2}\right]$$

$$= \frac{4R\sin\theta\cos\theta}{BD} + \frac{\cos^2\theta}{BD}\left[4H + \frac{6Ra}{D}\right] \tag{11.7}$$

For f to be maximum, $\frac{df}{d\theta} = 0$

Differentiating equation 11.7

$$\frac{r}{BD}\left[4R(\cos^2\theta - \sin^2\theta) - 2\cos\theta\sin\theta\left(4H + \frac{6Ra}{D}\right)\right] = 0$$

$$\text{or, } \tan 2\theta = \frac{2RD}{2HD + 3Ra} \tag{11.8}$$

Which gives the inclination of the plane of maximum bending stress.

Putting the above value of θ in equation 11.5 and 11.6, the values of direct pull and moment on the plane of worst stress may be obtained. The steel required to cater for both the direct pull and the moment may be determined from any of the available design charts.

Similarly, the critical plane for shear is determined as follows :

Let Φ be the angle of the critical plane with the vertical.

$$\therefore \text{ Shear force on the plane} = (R\cos\Phi + H\sin\Phi) \tag{11.9}$$

$$\text{Shear stress, } q = \frac{(R\cos\Phi + H\sin\Phi)}{BD\sec\Phi} \tag{11.10}$$

For q to be maximum, $\frac{dq}{d\Phi} = 0$

Differentiating equation 11.10 and equating it to zero,

$$\tan 2\Phi = \frac{H}{R} \tag{11.11}$$

The necessary shear reinforcement may be provided in the plane of maximum shear stress which may be worked out from equation 11.10 and 11.11.

ILLUSTRATIVE EXAMPLE 11.2

The vertical and horizontal loads on an articulation are 850 KN and 100 KN respectively. Design the reinforcement and show the details of the reinforcement for the articulation when D = 120 cm., a = 40 cm. and B = 75 cm.

Solution

Inclined Section

From equation 11.8,

$$\tan 2\theta = \frac{2RD}{2HD + 3Ra} = \frac{2 \times 850 \times 120}{2 \times 100 \times 120 + 3 \times 850 \times 40} = 1.6190$$

$\therefore 2\theta = 58^\circ - 18' \qquad \therefore \theta = 29^\circ - 09'$

From equation 11.5, $P = (R \sin\theta + H \cos\theta)$

$= (850 \times 0.4871 + 100 \times 0.8734) = 414.03 + 87.34 = 501.37$ KN

From equation 11.6, $M = R\left(a + \frac{D}{2}\tan\theta\right) + H \cdot \frac{D}{2} = 850\left(40 + \frac{120}{2} \times 0.5578\right) + 100 \times \frac{120}{2}$

$= 62{,}450 + 6{,}000 = 68{,}450$ KN cm.

With direct pull of 501.37 KN and moment of 68,450 KN cm. in the section, the percentage of steel is found from chart 68 of *"Design Aids to IS:456-1978"* as follows :

Assumptions :

i) Rectangular section with reinforcement equally divided on two sides.

ii) Cover 30 mm.

iii) $\frac{d'}{D} = \frac{30}{1200} = 0.025$

iv) Grade of concrete M20.

v) Grade of steel = S415.

vi) Factored pull = $1.75 \times 501.37 = 878$ KN

vii) Factored moment = $1.75 \times 68{,}450 = 1{,}19{,}800$ KN cm.

$$\therefore \frac{Pu}{fckBD} = (-)\frac{878 \times 10^3}{20 \times 750 \times 1200} = (-)0.049; \qquad \frac{Mu}{fckBD^2} = \frac{1{,}19{,}800 \times 10^4}{20 \times 750(1200)^2} = 0.055$$

From Chart 68, $\frac{p}{fck} = 0.045 \quad \therefore p = 0.045\,fck = 0.045 \times 20 = 0.9$

$$\therefore \text{Area of steel } = pBD = \frac{0.9}{100} \times 750 \times 1200 = 8100 \text{ mm}^2$$

Since reinforcement are provided at an angle of 45 degrees, the area of steel required to give an effective area of 8100 mm^2 steel is as below :

$$A_s = \frac{8100}{\cos(45^\circ - 29^\circ - 09')} = \frac{8100}{0.9629} = 8420 \text{ mm}^2$$

Using 12 nos. 32 Φ in two layers, area of steel provided = $12 \times 804 = 9600$ mm^2.

Shear in Inclined Plain

From equation 11.11, $\tan 2\Phi = \frac{H}{R} = \frac{100}{850} = 0.1176$

$\therefore 2\Phi = 6^\circ - 42'$ and $\Phi = 3^\circ - 21'$

From equation 11.9, Shear force = (R cos Φ + H sin Φ)

$$= (850 + 0.9982 + 100 \times 0.0585) = 854.32 \text{ KN}$$

$$\therefore \text{ Shear stress} = \frac{V}{BD \text{ Sec } \Phi} = \frac{854.32 \times 10^3}{750 \times 1200} = 0.95 \text{ MP}_a$$

This exceeds the allowable limit of shear stress without shear reinforcement (vide Table 5.12) i.e. 0.34 MP_a. Hence shear reinforcement is required. If 2 nos. 32 Φ bent up bars are provided, shear resistance = $2 \times 804 \times 200 \sin(45° - 3° - 21') = 2 \times 804 \times 200 \times 0.6646 = 213{,}700 \text{ N} = 213.7 \text{ KN}$

Balance shear = 854.32 – 213.7 = 640.62 KN

Using 12 Φ 6 legged stirrups @ 150 mm spacing, shear resisted by stirrups

$$= \frac{6 \times 113 \times 200 \times 1100}{150} = 994{,}400 \text{ N} = 994.4 \text{ KN}$$

This is more than balance shear of 640.62 KN; hence safe.

Moment and Shear in Vertical Plane

The direct pull and the moment may be obtained in the vertical plane putting the value of θ equal to zero in equation 11.5 and 11.6. The area required to be placed at 45° to get the effective steel area sufficient for resisting the above pull and moment may be found in the same manner as detailed in case of inclined section. The steel requred for the above is less than that for the inclined plane i.e., plane of maximum stress.

Beyond the neck, the inclined bars provided for resisting the pull and the moment will not be effective and therefore, additional bars are required to be provided. If calculated on the previous basis, the area of reinforcement required for the purpose comes to 5000 mm^2 and for this 7 nos. 32 Φ bars are necessary.

The shear in the vertical plane will be less than before and the reinforcement already provided for the plane of maximum stress will be sufficient.

The details of reinforcement in the articulation are indicated in Fig. 11.13.

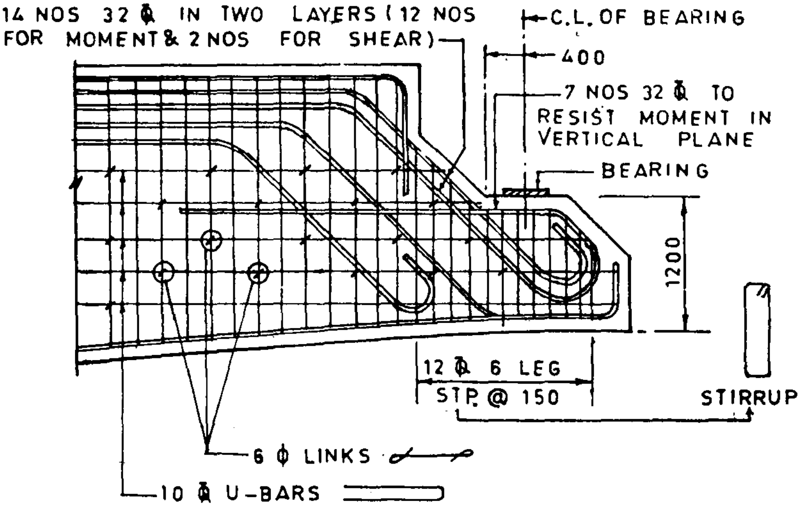


FIG. 11·13 DETAILS OF REINFORCEMENT IN ARTICULATION (Example 11·2)

11.7 REFERENCES

1. IRC : 21-1987 : Standard Specifications and Code of Practice for Road Bridges, Section III — Cement Concrete (Plain and Reinforced) (Second Revision) — Indian Roads Congress.
2. *"Design Aids for Reinforced Concrete to IS : 456-1978"* – Indian Standards Institution.
3. *"Highway Bridges in West Bengal"* – published by the Public Works and Public Works (Roads) Departments, Govt. of West Bengal, Calcutta, 1977.
4. Dayaratnam, P. — *"Design of Reinforced Concrete Structures (Second Edition)"* – Oxford & IBH Publishing Co., Calcutta 16.
5. Mallick, S.K. and Gupta, A.P. — *"Reinforced Concrete"* — Oxford & IBH Publishing Co., Calcutta.
6. Niyogi, S.K. and Murty, M.K.S. — *"Ultimate Strength of Bridge Articulation"* — Construction Engineers of India, Vol. VI, 1964, State Engineers' Association, Calcutta.
7. Rakshit, K. S. — *"A Study of the Economics of Solid Slab Balanced Cantilever Bridges"* — The Indian Concrete Journal, Bombay - 400020 (September, 1968 and January, 1970 issues).

CHAPTER 12

REINFORCED CONCRETE RIGID FRAME BRIDGES

12.1 GENERAL

12.1.1 In rigid frame bridges, the deck is rigidly connected to the abutments and piers. This type of structure may be a single span unit or a multi-span unit as indicated in Fig. 12.1. All the advantages of a continuous span bridge are present here. The following features are the additional advantages of the rigid frame bridges over the continuous ones.

i) More rigidity of the structure.
ii) Less moments in deck being partly transferred to the supporting members.
iii) No bearings are required.
iv) Better aesthatic appearance than the continuous span structure.

As in continuous span bridges, these structures also require unyielding foundation materials. The analysis is, however, more laborious than the former.

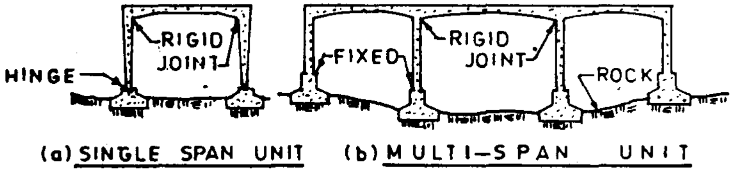


FIG. 12·1 RIGID FRAME STRUCTURES

12.1.2 The frames may be hinged or fixed at the base as illustrated in Fig. 12.1. When hinged, the moments carried over to the base rotate only the vertical supports thereby reducing the moments very considerably and no moments are carried over to the footings; only the vertical load and the moment caused by the thrust at the hinge level are to be considered in designing the footings. In fixed base structures, on the other hand, the moments from the superstructure are ultimately carried over to the footings since the vertical supports cannot rotate independently without rotating the footings along with them. It is, therefore, evident that in hinged frames, the moments at the base of supports and at rafts are very much less but the span moments are greater than those of fixed frames. Since the fixed frames are designed on the assumption that the vertical members do not rotate at the base it is possible to achieve this state of condition only if the foundation can rest on solid rock or un-yielding foundation.

12.2 TYPES OF RIGID FRAME BRIDGES

12.2.1 A few types of rigid frame bridges have been illustrated in Fig. 4.5 and 4.6. Solid slab rigid frame bridges upto 25 m span may be possible while slab and girder type rigid frames may be used upto span of 35 m. In road overbridges, the cantilever type of portal frames as indicated in Fig. 4.6 is commonly favoured.

12.2.2 Rigid frame box culverts or minor bridges (single or multiple Fig. 4.5) are usually adopted in areas where the foundation soil is weak and wider foundation area is desirable for bringing down the foundation pressure within safe values permissible for the type of soil.

12.3 PROPORTIONING OF STRUCTURES

12.3.1 The ratio of intermediate to end span of rigid frame bridges should be as follows :

For slab bridges	1.20 to 1.30
For slab and girder bridges	1.35 to 1.40

For rough estimate of the section, the dimensions of the mid span and support section for solid slab bridges may be taken as $\frac{L}{35}$ and $\frac{L}{15}$ respectively.

12.3.2 The soffit curves for rigid frame bridges are generally made the same as those for continuous bridges viz. straight and parabolic haunches about which detailed discussions have been made in Chapter 10.

12.4 METHOD OF ANALYSIS AND DESIGN CONSIDERATIONS

12.4.1 In analysing rigid frame structures, the method of moment distribution is commonly employed. As already mentioned in Chapter 10 in dealing with continuous bridges, moment distribution method is best suited for practical design because the sections of the structures vary at different points for which other methods are laborious and therefore, unsuitable. If the values of stiffness factors, carryover factors and fixed end moments for different joints of a rigid frame structure are known, the use of the moment distribution method is very simple.

12.4.2 Since the deck of the rigid frame bridges is connected to the supporting members rigidly, the following considerations are to be made in designing such structures in addition to dead load and live load effects.

12.5 TEMPERATURE EFFECT

12.5.1 The rise or fall of temperature causes elongation or contraction of decks which gives rise to fixed end moments on the vertical members as explained hereinafter (Fig. 12.2).

12.5.2 Elongation or contraction of deck BC due to temperature variation of $t = \delta_2 = L_2 \alpha t$.

Elongation or contraction of deck AB or CD due to temperature variation of $t = \delta_1 = L_1 \alpha t$ but due to elongation or contraction of deck BC by δ_2, the nett movement of A or C will be $(\delta_1 + \frac{1}{2}\delta_2)$.

The fixed end moment on a vertical member having moment of inertia, I and deflection, δ, may be given by

$$FEM = \frac{6\,EI\delta}{(L)^2} \qquad (12.1)$$

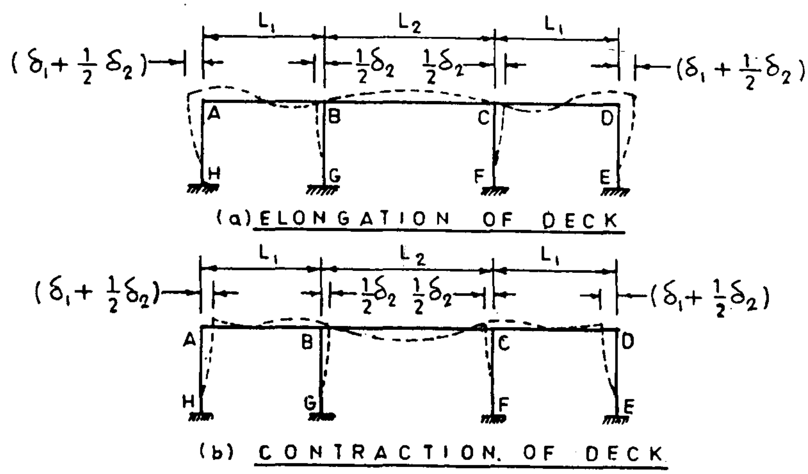


FIG. 12·2 EFFECT OF TEMPERATURE ON RIGID FRAME

12.5.3 The fixed end moments so developed on the top and bottom of all vertical members as per equation 12.1 may be distributed over all the members.

12.6 EFFECT OF SHRINKAGE, WIND, SEISMIC AND WATER CURRENT

12.6.1 Due to shrinkage of concrete, the deck contracts causing thereby the same nature of effect as the fall of temperature does. Normally, the effect due to shrinkage is assumed as equivalent in magnitude to that produced by the fall of temperature.

12.6.2 The wind blowing at an inclination to the piers may give rise to sway moments which will be shared by all the members of the frame after distribution.

12.6.3 The seismic force acting at deck, piers and abutments will cause moments in the members of the frame as wind force will induce.

12.6.4 The cross current flowing through the river strikes the piers and abutments and this will induce moments on the members as the wind will do.

12.7 DESIGN PROCEDURE

1. Select span lengths for end and intermediate spans appropriate to the site conditions and type of bridges. The depths at mid-span and at supports are to be assumed.
2. Select the soffit curve and find the depths at various sections. Calculate the fixed end moments due to uniformly distributed dead load and haunch load from standard design tables such as *"The Applications of Moment Distribution"*, published by The Concrete Association of India, Bombay.
3. Find the values of stiffness factors and carryover factors from design tables after evaluating the values of frame constants such as a_A, a_B, r_A, r_B, h_C etc. The distribution factors may be determined as follows:

$$D_{AB} = \frac{S_{AB}}{\Sigma S}$$

Where D_{AB} = distribution factor for member AB.

S_{AB} = Stiffness factor for AB.

$\sum S$ = Sum of the stiffness factors of all the members of that joint.

4. The dead load fixed end moments are to be distributed and sway correction made if required.
5. To evaluate the live load moments on the members, influence line diagram for each member has to be drawn. The procedure will be laborious if the moments are to be obtained by placing unit load on each section (there may be 5 to 10 sections on each span depending on the span length) and distributing the fixed end moments due to unit load with sway correction where necessary. The method may be simplified if the procedure given below is followed.
6. Place the unit load at any position (Fig. 12.3) and obtain the fixed end moments x and y at end B and C. Distribute these fixed end moments over all the members. The moments so obtained at various sections are the live load moments (elastic) due to the unit load under consideration. After necessary sway correction, the moment equation in terms of x and y will give the ordinate of the bending moment influence line diagram at various sections for that unit load. Now, from the tables or graphs, the values of x and y for unit load at different load positions may be known from which the ordinates of the influence line diag. at various sections for different load position may be calculated. The procedure outlined above will require one set of moment distribution and one set of sway correction of the moment equations for each span.

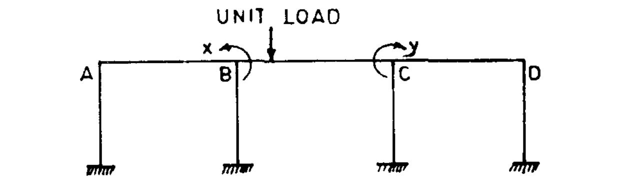


FIG. 12·3 FIXED END MOMENTS DUE TO UNIT LOAD

The influence line diagram obtained by the method described will be only for elastic moment. The free moment diagram will have to be superimposed over it to get the nett influence line diagram. The live load moments may thereafter be obtained from the influence line diagram.

7. Work out the moments on various members and at various sections due to temperature, shrinkage, wind, water currents, earth pressure on abutments, seismic force etc.
8. The moments obtained due to various loadings and effects as enumerated above may be summed up in such a way that the design moments are maximum for all possible combination cases.
9. Check the adequacy of the sections in respect of concrete stresses and provide necessary reinforcement to cater for the design moment.
10. Detail out the reinforcement properly.

12.8 REFERENCES

1. *"Concrete Bridges"*— The Concrete Association of India, Bombay 20.
2. Reynolds, C.E. — *"Reinforced Concrete Designers' Handbook (Fifth Edition)"*— Concrete Publications Ltd., 14, Dartmouth Street, London SW1

Chapter 13

REINFORCED CONCRETE ARCH BRIDGES

13.1 GENERAL

13.1.1 Reinforced concrete arch bridges are adopted when girder bridges prove to be uneconomic. With the increase in span, the section of the girder increases to such an extent that the self weight of the girders becomes a substantial part of the total loads. Compared to the girder bridges, arch bridges are economic because the

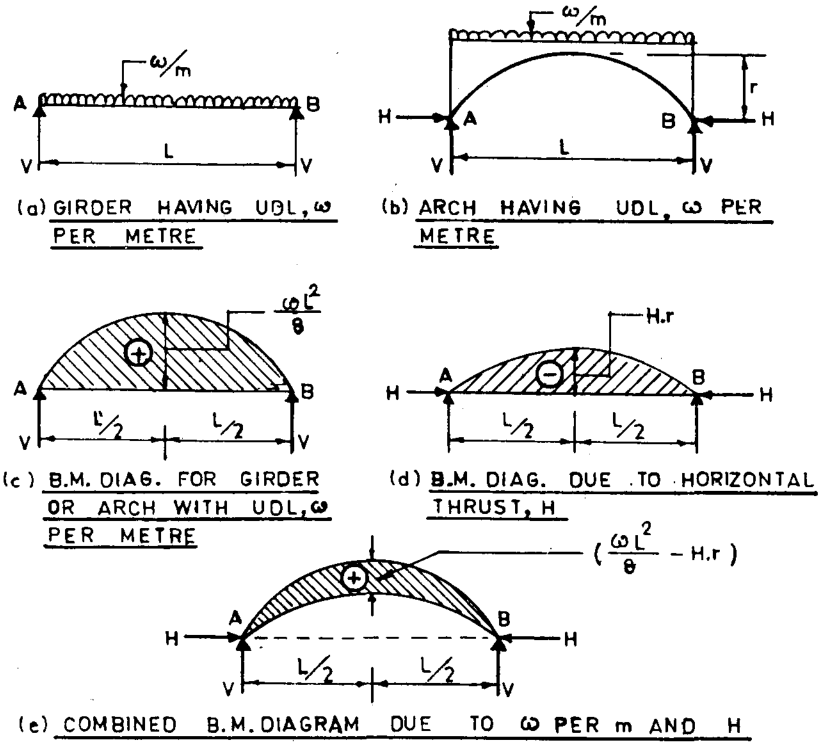


FIG. 13·1

dead load moments in an arch bridge are almost absent when the arch is properly designed. This is illustrated in Fig. 13.1.

13.1.2 An arch is a structural member curved in a vertical plane and the loads on the arch are carried by the arch ribs mainly through direct axial thrusts, the bending moments and shear forces being small compared to a girder which requires larger section to withstand larger bending moments and shear forces caused by the same loading. This is due to the fact that while a simply supported girder will have only the sagging (positive) moment on account of external loads, an arch, on the other hand, will have not only the same sagging moment but will also have a hogging (negative) moment of opposite nature to partly balance the sagging moment thereby reducing the sagging moment to a considerable extent. The hogging moment is generated by a horizontal force, H, at the support due to the shape of the arch as in a portal frame (see Fig. 13.1).

13.1.3 The main parameter of an arch bridge is the ratio of the rise to the span, $\frac{r}{L}$. This ratio varies from $\frac{1}{6}$ to $\frac{1}{10}$ depending upon the site conditions and the surroundings. The greater is the ratio, the lesser is the thrusts on the supports.

From the consideration of economy, it is attempted to coincide the centre of pressure of a given load with the centre line of the arch. The moment of an arch is given by —

$$M = M_1 - H \cdot y \tag{13.1}$$

Where
- M = Arch moment at any section, x
- M_1 = Moment considering the arch as a simply supported beam
- H = Horizontal force at the springing
- y = Vertical ordinate of the arch centre at section x from the springing

The configuration of the centre of pressure in the arch is obtained from equation 13.1 assuming that M = 0, i.e.,

$$y = \frac{M_1}{H} \tag{13.2}$$

13.1.4 It is not possible in practice to attain a complete coincidence of the arch axis with the centre of pressure since the arch is subjected to live loads of various distribution which requires to check the design under worst condition of loading in addition to dead loads, temperature variations and the effect of creep and shrinkage etc. Therefore, attempts are made to achieve the lowest values of the design forces and moments as far as possible.

13.1.5 Since the arch ribs are subjected to direct axial thrust and moment, they are designed on the basis of section subjected to eccentric compression. The rib section may be a rectangular or a T-section. Reinforcement are provided in both the faces of the section since moment of opposite sign may occur at the section due to various combination of loadings.

13.2 TYPES OF ARCH BRIDGES

13.2.1 The arch bridges may be classified from two considerations as below :

(a) Location of deck with respect to the arch rib (Fig. 13.2)

i) Deck type

ii) Through type

iii) Semi-through type

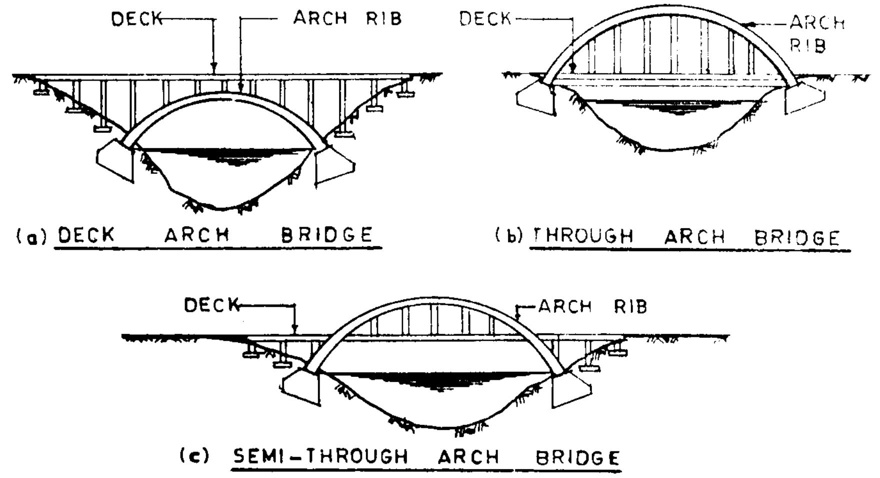


FIG. 13·2 TYPES OF ARCH BRIDGES BASED ON LOCATION OF DECK

(b) Structural arrangement of arch rib (Fig. 13.3)

i) Two hinged arch

ii) Three hinged arch

iii) Fixed arch

iv) Tied arch or bow-string girder.

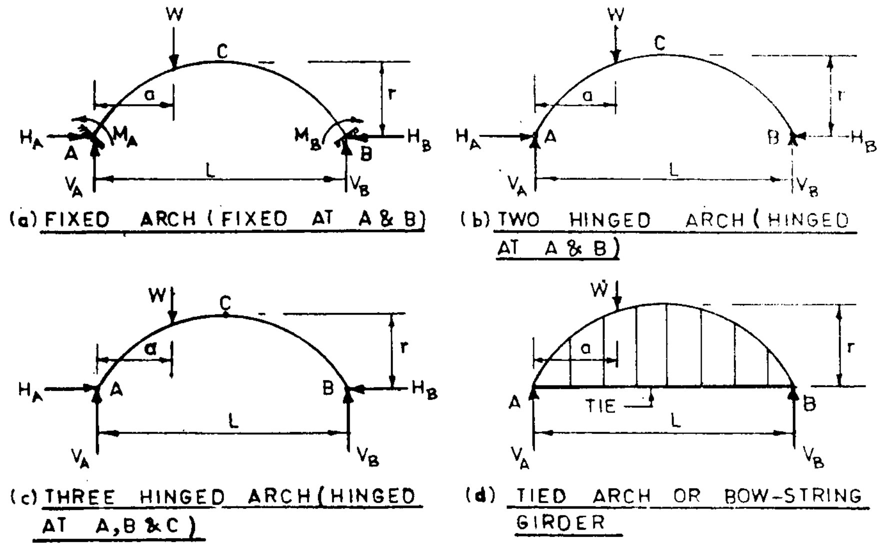


FIG. 13·3 TYPES OF ARCHES BASED ON SUPPORTING ARRANGEMENT

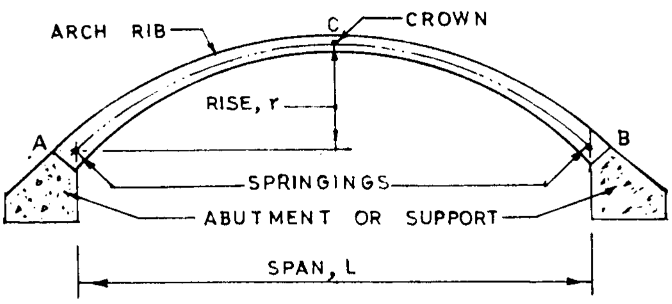


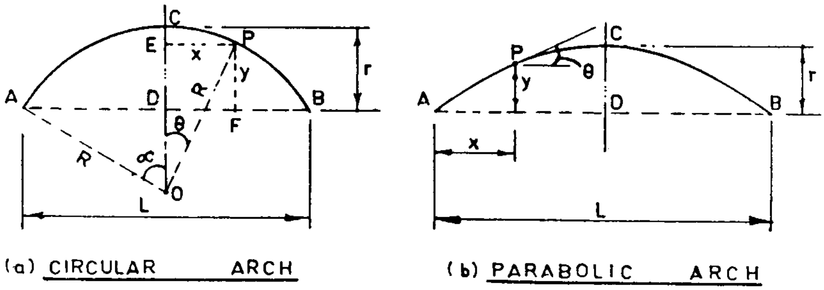
FIG. 13·4 COMPONENTS OF ARCHES

13.3 MAIN COMPONENTS OF AN ARCH

13.3.1 One fixed arch is shown in Fig. 13.4 in which A and B are abutments or supports where the arch rib is fixed. In case of two hinged, the arch rib is hinged at A and B. For a three hinged arch, a third hinge is provided at C in addition to two hinges at A and B. The junction of the arch rib with the abutments is known as *"Springing"* and the top-most part of the arch rib is the *"crown"*. In case of tied arches, both the springings of the arch are connected by a tie and while one springing is hinged at the abutment, the other springing is supported on the other abutment through movable rollers.

13.4 SHAPE OF ARCH BRIDGES

13.4.1 The arches are generally circular or parabolic as shown in Fig. 13.5.



(a) CIRCULAR ARCH (b) PARABOLIC ARCH

FIG. 13·5 VARIOUS ARCH SHAPES

13.4.2 Properties of a Circular Arch

Referring to Fig. 13.5a, OA = OB = OC = OP = R (Radius of the arch); AB = L (Span of the arch); CD = r (Rise of the arch); x & y are co-ordinates of P from origin D.

In the right-angled triangle OEP,

$$OP^2 = OE^2 + EP^2 \quad \text{i.e.} \quad R^2 = (R - r + y)^2 + x^2 \tag{13.3}$$

Equation 13.3 gives the relationship of R with x & y.

$$\text{Also } x = OP \sin\theta = R \sin\theta \tag{13.4}$$

$$\text{And } y = OE - OD = R\cos\theta - R\cos\alpha \quad = R(\cos\theta - \cos\alpha) \tag{13.5}$$

It is known that in a segment of a circle, $(2R - r)\,r = \frac{L^2}{4}$

$$\text{or, } 2R = \frac{L^2}{4r} + r \quad \text{i.e.} \quad R = \frac{L^2}{8r} + \frac{r}{2} \tag{13.6}$$

$$\text{Also } \sin\alpha = \frac{AD}{A\Theta} = \frac{L}{2} \div R = \frac{L}{2R} \tag{13.7}$$

$$\text{And } \cos\alpha = \frac{OD}{AO} = \frac{(R - r)}{R} \tag{13.8}$$

13.4.3 Properties of a Parabolic Arch

Referring to Fig. 13.5b, AB = L (Span of the arch); CD = r (Rise of the arch); x & y are co-ordinates of P from origin A. The equation of parabola is given by,

$$y = Kx\,(L - x) \tag{13.9}$$

Where K is a constant

When $x = \frac{L}{2}$, $y = r$. Substituting these values of x & y in equation 13.9, we get $r = K \cdot \frac{L}{2}\left(L - \frac{L}{2}\right)$ or, $K = \frac{4r}{L^2}$

Putting this value of K, equation 13.9 becomes

$$y = \frac{4rx}{L^2}(L - x) \tag{13.10}$$

Equation 13.10 gives the rise of the arch rib from the springing at a distance x from the springing.

The slope of the arch rib at x may be obtained by differentiating equation 13.10.

$$\text{Slope of arch rib} = \tan\theta = \frac{dy}{dx} = \frac{4r}{L^2}(L - 2x) \tag{13.11}$$

13.5 DISTINCTIVE FEATURES OF VARIOUS ARCHES

13.5.1 As described in Art. 13.2, arches may be fixed, hinged or tied at the supports. Due to the curved shape of an arch, horizontal forces are developed at the supports in addition to vertical forces both in the fixed and the hinged arches. For fixed arches, fixing moments are also generated at the supports. It is stated in Art. 13.1.2 that the horizontal forces at the supports produce hogging moments at all sections of the arch, and thereby reduce the sagging moments resulting in reduced cross section of the arches compared to the girders.

13.5.2 In two and three hinged arches, only the thrusts are transmitted to the supports or abutments and there is no bending moment on the arch at the springing (refer Fig. 13.2). In case of a fixed arch, however, there will be fixing moments at the supports in addition to the thrusts. Forces and moments in fixed arches change both due to rotation and displacement of the supports and therefore, fixed arches are constructed where absolute un-yielding foundation condition is available. In case of two hinged arches, the structure is not affected due to rotation of the abutments but is affected due to the displacement of the same. Therefore, two hinged arches may be designed with small displacement of the supports. The case is much better for a three hinged arches so far as the rotation and displacement of the foundation are concerned. Even with rotation and small displacement of the foundation or unequal settlement of the foundations, the thrusts and moments are not significantly affected in three hinged arch bridges.

13.6 FORCES AND MOMENTS ON ARCH BRIDGES

13.6.1 Forces and Moments due to Dead Loads and Superimposed Loads

13.6.1.1 All types of arch ribs will be subjected to thrusts and moments due to dead and superimposed loads. The abutments will also be subjected to thrusts and moments in case of fixed arches only but hinged arches will have only thrusts and no moments at the abutments.

13.6.2 Forces and Moments due to Temperature Variation

13.6.2.1 In addition to the thrusts and moments due to dead and superimposed loads, rise of temperature will cause thrusts and moments and fall of temperature will cause pull and moments in the arch ribs of all types of arches. For fall of temperature, the abutments will get pull and hogging moment in fixed arches but pull and sagging moment in hinged arches. For concrete arches, the effective temperature variation is generally taken as two-third of the actual temperature variation.

13.6.3 Forces and Moments due to Arch Shortening

13.6.3.1 Arch shortening or rib shortening is caused due to the compressive strain of the arch concrete by the direct axial thrust in the rib on account of external loading on the arch rib. This phenomenon releases part of the horizontal thrust produced by the dead and superimposed loads.

13.6.4 Forces and Moments due to Shrinkage of Concrete

13.6.4.1 Shrinkage of concrete shortens the length of the arch rib and its effect on the arch is similar to that due to fall of temperature. Shrinkage is more at the initial stage but its quantum is gradually reduced as the concrete hardens. Shrinkage is minimised by adopting high grade concrete in arches. It can further be reduced by pouring concrete in arch ribs in sections leaving gaps at the crown and the springings which are concreted later on.

13.6.5 Forces and Moments due to Plastic Flow of Concrete

13.6.5.1 Plastic flow or creep of concrete is a phenomenon which causes a permanent strain in the concrete when loaded for a long time. Similar to the shrinkage strain, creep strain is more at the initial stage and then becomes less and less as time passes. The plastic flow of concrete causes pull and hogging moments at the supports in fixed arches while it causes pull and sagging moments at the supports in hinged arches. Similar to the fall of temperature or shrinkage in concrete, plastic flow can be minimised by using high grade concrete in arch ribs.

13.7 ANALYSIS OF ARCH BRIDGES

13.7.1 Effect of Dead Loads & Superimposed Loads

13.7.1.1 ***Two-hinged Arches***

A two-hinged arch has four unknown reaction components at the two supports viz. H_A, V_A at support A and H_B, V_B at support B as shown in Fig. 13.3b. Using three important equations of statics we get —

i) $\Sigma H = 0$ i.e. $H_A + H_B = 0$ i.e. $H_A = (-)H_B = H$ (say) **(13.12)**

ii) $\Sigma V = 0$ i.e. $V_A + V_B - W = 0$ i.e. $V_A + V_B = W$ **(13.13)**

iii) $\Sigma M = 0$; taking moment about A,

$$(V_B \cdot L - W \cdot a) = 0 \text{ or, } V_B = \frac{Wa}{L}$$

∴ From equation 13.13,

$$V_A = W - V_B = W - \frac{Wa}{L} = \frac{W(L-a)}{L} \tag{13.14}$$

From equation 13.1, moment at any section of the arch rib is given by $M = M_1 - Hy$. Hence, if the magnitude of H is known the values of all the four unknown reaction components may be obtained and M_x at any section of the arch rib will also be known.

Since there are four unknown reaction components and three known equations of statics, the structure is indeterminate to the first degree. The fourth equation can be framed from displacement consideration. It is known from Castigliano's First Theorem that the partial derivative of the total strain energy in any structure with respect to the applied force or moments gives the displacement or rotation respectively at the point of application of the force or the moment in the direction of the applied force or moment. Therefore, if the supports do not yield, the partial derivative of the total strain energy with respect to the horizontal thrust will be zero.

If the supports yield by an amount δ in the direction of the horizontal thrust, then the partial derivative of total strain energy with respect to the horizontal thrust will be equal to δ.

From equation 13.1, $M = M_1 - H \cdot y$. Neglecting strain energy due to direct thrust which is small, total strain energy due to bending moment will be

$$U = \int_o^L \frac{M^2}{2EI}\,ds = \int_o^L \frac{(M_1 - Hy)^2}{2EI}\,ds \tag{13.15}$$

If the supports do not yield, $\frac{\partial U}{\partial H} = 0$

$$\therefore \frac{\partial U}{\partial H} = \int_o^L \frac{(M_1 - Hy)(-y)}{EI}\,ds = 0$$

$$\therefore H = \frac{\int_o^L \frac{M_1\ yds}{EI}}{\int_o^L \frac{y^2\ ds}{EI}} \quad \text{i.e.}\ H = \frac{\int_o^L M_1\ yds}{\int_o^L y^2\ ds} \tag{13.16}$$

In case the supports yield, then $\frac{\partial U}{\partial H} = (-)\,\delta$ i.e. $\int_o^L \frac{(M_1 - Hy)(-y)\,ds}{EI} = (-)\,\delta$

$$\therefore H = \frac{\int_o^L \frac{M_1\ yds}{EI} - \delta}{\int_o^L \frac{y^2\ ds}{EI}} = \frac{\int_o^L M_1\ yds - EI\delta}{\int_o^L y^2\ ds} \tag{13.17}$$

Normally the moment of inertia of the arch rib at any section varies as the secant of the angle θ at the section and as such $I = I_c \sec\theta$ where I_c is the moment of inertia at the crown section.

Also $ds = dx \sec\theta$

In such case of variable moment of inertia of arch sections, equation 13.16 and 13.17 change to equation 13.18 and 13.19 respectively as below :

$$H \text{ (for no support yield)} = \frac{\int_0^L M_1\, y\, dx}{\int_0^L y^2\, dx} \tag{13.18}$$

$$H \text{ (for support yield of } \delta) = \frac{\int_0^L M_1\, y\, dx - EI_c\delta}{\int_0^L y^2\, dx} \tag{13.19}$$

Therefore, as stated before, when the value of H is known either from equation 13.18 or 13.19 as the case may be, all forces and moments of the arch structure can be found out.

13.7.1.2 *Three-hinged Arch*

As in two-hinged arch, three-hinged arches have also four unknown reaction components viz., H_A, V_A, H_B & V_B as shown in Fig. 13.3c. But since these arches have a third hinge at the crown when $M_c = 0$, three-hinged arches are statically determinate having the fourth equation viz., $M_c = 0$. Forces and moments on the arch are determined as below :

i) $\Sigma H = 0$. i.e. $H_A + H_B = 0$, i.e. $H_A = (-)\, H_B = H$ (say).

ii) $\Sigma V = 0$, i.e. $V_A + V_B = W$.

iii) $\Sigma M = 0$, ∴ Taking moment about A

$$(V_B \cdot L - Wa) = 0 \text{ or } V_B = \frac{Wa}{L} \tag{13.20}$$

$$\text{And } V_A = W - V_B = W - \frac{Wa}{L} = \frac{W(L-a)}{L} \tag{13.21}$$

iv) $M_C = 0$. ∴ Taking moment about C from equation 13.1,

$M_C = M_1 - Hr = 0$

$$\text{or } H = \frac{M_1}{r} \tag{13.22}$$

$$\text{Where } M_1 = V_A \cdot \frac{L}{2} - W\left(\frac{L}{2} - a\right) = \frac{W(L-a)}{L} \cdot \frac{L}{2} - W\left(\frac{L}{2} - a\right)$$

Therefore, all forces and moment at any section of the three hinged arch can be evaluated.

13.7.1.3 *Fixed Arches*

From Fig. 13.3a, it may be noted that there are six unknown reaction components at the two supports viz. H_A, V_A, M_A at support A and H_B, V_B, M_B at support B. As mentioned in case of two and three hinged arches in Art. 13.7.1.1 and Art. 13.7.1.2, only three equations of statics are available for the solution of unknown terms. Therefore, the fixed arch is statically indeterminate to the third degree. As in Art. 13.7.1.1, Castigliano's First Theorem may be made use of in framing the other three equations from the considerations that the rotation as well as the vertical and horizontal displacements at the supports are zero. Castigliano's First Theorem states that the partial derivative of the total strain energy in any structure with respect to the applied force or moments gives the displacement or rotation respectively at the point of application of the force or moments in the direction of the applied force or moments. Therefore, these three additional equations may be framed as under taking total strain energy, U of the arch as

$$U = \int_o^L \frac{M^2\,ds}{2\,EI} \tag{13.23}$$

i) No horizontal displacement of the abutments,

$$U = \int_o^L \frac{M^2\,ds}{2\,EI} = \int_o^L \frac{M^2\,dx}{2\,EI_C}$$

[Since $ds = dx \sec\theta$, and $I = I_C \sec\theta$]

$$\therefore \frac{\partial U}{\partial H} = \frac{1}{EIc}\int_o^L M\cdot\frac{\partial M}{\partial H}\,dx = 0 \tag{13.24}$$

ii) No vertical displacement of the abutments,

$$\therefore \frac{\partial U}{\partial V} = \frac{I}{EI_c}\int_o^L M\,\frac{\partial M}{\partial V}\,dx = 0 \tag{13.25}$$

iii) No rotation of the abutments,

$$\therefore \frac{\partial U}{\partial M} = \frac{1}{EIc}\int^L M\cdot\frac{\partial M}{\partial M}\,dx = 0 \tag{13.26}$$

By solving these three simultaneous equations from 13.24 to 13.26, the forces and moments of a fixed arch can be obtained.

13.7.1.4 *Elastic Centre for Fixed Arches*

In a two-hinged arch, the origin of the cordinates may be considered at one of the abutments but such assumption in case of a fixed arch involves much laborious works. The solution of simultaneous equations involving H, V and M determined from equations 13.24 to 13.26 for fixed arches is also a time consuming process. The analysis of fixed arches, on the other hand, can be conveniently done lay *"Elastic Centre Metho"*

The elastic centre is a point say, O, just below the crown (Fig. 13.6a) which is the centre of gravity of the factors $\frac{ds}{EI}$ for the various 'ds' elements of the arch axis. This factor is termed as *'Elastic Weight'* and the point 'O' as the *'Elastic Centre'* of the arch. The co-ordinates of the elastic centre are given by :

$$x_o = \frac{\int \frac{xds}{EI}}{\int \frac{ds}{EI}} = \frac{\int xds}{\int ds} \tag{13.27}$$

$$y_o = \frac{\int \frac{yds}{EI}}{\int \frac{ds}{EI}} = \frac{\int yds}{\int ds} \tag{13.28}$$

In case of symmetrical arches, x_o coincides with the vertical line passing throu[illegible] crown, i.e., the elastic centre will lie below the crown and on the vertical line passing through the cro[illegible].

Therefore, $x_o = \frac{L}{2}$

And if $I = I_c \sec\theta$ and $ds = dx \sec\theta$, then

$$y_o = \frac{\int ydx}{\int dx} \tag{13.29}$$

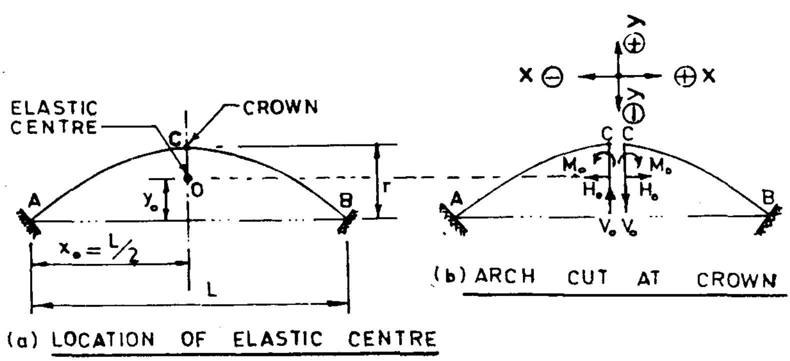


FIG. 13·6 ELASTIC CENTRE OF ARCH (Jai Krishna et al)[2]

The fixed arch is analysed by the Elastic Centre method by cutting the arch section at the crown., C and connecting the crown, C and the elastic centre, O by rigid arm CO, as shown in Fig. 13.6b. The bending moment M at any section of the two halves of the arch having co-ordinates (x, y) with reference to the elastic centre, O are given by :

For the left half, $M = (-)V_o x - H_o y + M_o + M_1$

For the right half $M = V_o x - H_o y + M_o + M_2$

Where M_1 and M_2 are the moments due to external loads.

From equation 13.24, $\dfrac{\partial U}{\partial H_o} = \dfrac{1}{EIc}\displaystyle\int_o^L M \dfrac{\partial M}{\partial H_o}\, dx = 0$

$$\text{or, } \frac{1}{EIc}\int_{-\frac{L}{2}}^{o} (-V_o x - H_o y + M_o + M_1)(-y)\, dx + \frac{1}{EIc}\int_{o}^{+\frac{L}{2}} (V_o x - H_o y + M_o + M_2)(-y)\, dx = 0 \qquad (13.30)$$

Since the origin has now been shifted to O, the elastic centre, the terms involving :

$\displaystyle\int_{-\frac{L}{2}}^{\frac{L}{2}} x dx$, $\displaystyle\int_{-\frac{L}{2}}^{\frac{L}{2}} y \cdot dx$ and $\displaystyle\int_{-\frac{L}{2}}^{\frac{L}{2}} xy dx$ will be zero. This simplifies equation 13.30 as below :

$$\frac{1}{EIc}\left[\int_{-\frac{L}{2}}^{o} (-H_o y + M_1)(-y)\, dx + \int_{o}^{\frac{L}{2}} (-H_o y + M_2)(-y)\, dx\right] = 0$$

$$\therefore H_o = \frac{\displaystyle\int_{-\frac{L}{2}}^{o} M_1 y dx + \int_{o}^{\frac{L}{2}} M_2 y dx}{2\displaystyle\int_{o}^{\frac{L}{2}} y^2 dx} \qquad (13.31)$$

Similarly $\dfrac{\partial U}{\partial V_o} = \dfrac{1}{EIc}\displaystyle\int_o^L M \dfrac{\partial M}{\partial V_o}\, dx = 0$

$$\therefore V_o = \frac{\displaystyle\int_{o}^{\frac{L}{2}} M_1 x dx + \int_{o}^{\frac{L}{2}} M_2 x dx}{2\displaystyle\int_{o}^{\frac{L}{2}} x^2 dx} \qquad (13.32)$$

and $\dfrac{\partial U}{\partial M_o} = \dfrac{1}{EIc}\displaystyle\int_o^L M \dfrac{\partial M}{\partial M_o}\, dx = 0$

$$\therefore M_o = \frac{(-)\int_{-\frac{L}{2}}^{o} M_1 dx - \int_{o}^{+\frac{L}{2}} M_2 dx}{2\int_{o}^{\frac{L}{2}} dx} \tag{13.33}$$

It may be noted that the numerator of equation 13.31 is the *"sum or integration of y times the free bending moments caused by both left hand and right hand loads"*. Similarly equation 13.32 is the *"sum or integration of x times the free bending moments of both left and right hand loads"* and equation 13.33 is the *"sum or integration of the free bending moments of the left and right hand loads"*. This shows that by shifting of the origin to the elastic centre, the values of the statically indeterminate forces and moments can be found directly without the solution of simultaneous equations.

It is also mentioned here that the forces and moments on the abutments may be evaluated from H_o, V_o and M_o as shown in the following illustrative example.

ILLUSTRATIVE EXAMPLE 13.1

Calculate the thrusts and moments at both the abutments of the fixed parabolic arch shown in Fig. 13.7 making use of the Elastic Centre method using equations 13.31 to 13.33.

Given,

(a) E is constant. (b) Moment of inertia varies as the secant of the slope.

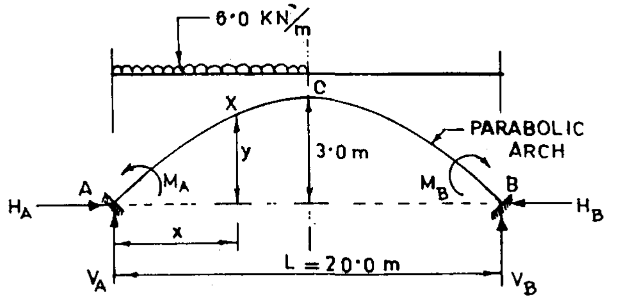


FIG.13·7 DETAILS OF ARCH (Example 13·1)

Analysis of the fixed arch by Elastic Centre Method using equations 13.31 to 13.33.

y_o = Height of elastic centre of a parabolic arch from springing $= \frac{2}{3} \times$ rise.

In the arch under consideration, $y_o = \frac{2}{3} \times 3.0 \text{ m} = 2.0 \text{ m}$

The equation of a parabola with elastic centre as the origin is given by $y = ax^2 + b$, where a and b are constants.

When $x = 0$, $y = 1.0$ m; $\therefore b = 1$.

When $x = 10.0$ m, $y = (-)\ 2.0$ m; $\therefore a = (-)\frac{3}{100} = (-)\ 0.03$

∴ The equation of the parabola becomes :

$$y = (1 - 0.03\,x^2) \tag{13.34}$$

From equation 13.31, $H_o = \dfrac{\int_{-\frac{L}{2}}^{o} M_1 y dx + \int_{o}^{\frac{L}{2}} M_2 y dx}{2\int_{o}^{\frac{L}{2}} y^2 dx}$

M_1 on the left side at a section $x = \dfrac{6x^2}{2} = 3x^2$

M_2 on the right side at $x = 0$ since there is no loading. Putting the value of y from equation 13.34,

$$H_o = \frac{\int_{-10}^{o} 3x^2 (1 - 0.03x^2)\, dx}{2\int_{o}^{10} (1 - 0.03x^2)^2\, dx} = 50 \text{ KN}$$

From equation 13.32, $V_o = \dfrac{\int_{-\frac{L}{2}}^{o} M_1 x dx + \int_{o}^{\frac{L}{2}} M_2\, x dx}{2\int_{o}^{\frac{L}{2}} x^2 dx}$

As before M_1 and M_2 are $3x^2$ and zero respectively, ∴ $V_o = \dfrac{\int_{-10}^{o} 3x^3 \cdot dx}{2\int_{o}^{10} x^2 dx} = 11.25$ KN

From equation 13.33, $M_o = \dfrac{(-)\int_{-\frac{L}{2}}^{o} M_1 dx - \int_{o}^{\frac{L}{2}} M_2 dx}{2\int_{o}^{\frac{L}{2}} dx}$

As before $M_1 = 3x^2$ and $M_2 = 0$, $\therefore$ $M_o = \dfrac{(-)\int_{-10}^{o} 3x^2 dx}{2\int_{o}^{10} dx} = 50$ KNm.

The values of H_o, V_o and M_o are at the elastic centre from which the forces and moments on the abutments may be evaluated as under :—

Since there is no load on the right half,

$H_B = H_o = 50$ KN ; $V_B = V_o = 11.25$ KN ; and $H_A = H_B = 50$ KN

V_A = Total load $- V_B = 60.0 - 11.25 = 48.75$ KN

Taking moment about A,

$M_A - \frac{6 \ 10^2}{2} + V_o x\ 10 + H_o x\ 2 + M_o = 0$; or, $M_A = 300 - 112.5 - 100 - 50 = 37.5$ KNm

Similarly, $M_B - V_o \times 10 + H_o \times 2 + M_o = 0$; or, $M_B = 112.5 - 100 - 50 = (-)\ 37.5$ KNm, i.e., anti-clockwise.

The forces and moments at the abutments by both the methods can be determined but it is evident that analysis of the fixed arch by the elastic centre method is much less laborious than by solving the simultaneous equations.

13.7.1.5 *Tied Arches*

Tied arches are modified two-hinged arches. In two-hinged arches, the horizontal thrusts are resisted by the abutments whereas in tied arches, the horizontal thrusts are resisted by a tie provided at the springing level. Due to external loading on the arch, the springing points of the arch tend to move outwards which is prevented by the tie partially. The tie, being in tension, is subjected to tensile deformation which allows one end of the arch provided with rollers to move such that the outward force of the arch at the springing level balances the tension in the tie. For the stability of the tied arch, one end of the arch at the springing level is provided with a hinge and the other end with a roller. The tensile deformation of the tie allowing the free end of the tie to move reduces the magnitude of the horizontal force at the support compared to a two-hinged or fixed arch wherein the displacement of the arch ends is prevented. It is needless to mention that the tension in the tie is the horizontal force on the arch ends.

As in two-hinged arches, tied arches will have four unknown reaction components viz. H_A, V_A, H_B and V_B for which three equations are available from statics, i.e. $\Sigma H = 0$, $\Sigma V = 0$ and $\Sigma M = 0$, the fourth equation is $\frac{\partial U}{\partial H} = 0$ for two hinged arches but in case of tied arches, $\frac{\partial U}{\partial H} \neq 0$ as arch end moves. Therefore, this equation cannot be used. Since displacement of the supports in the vertical direction is zero, this consideration may be utilised in framing the fourth equation viz. $\frac{\partial U}{\partial V} = 0$.

13.8 FORCES AND MOMENTS DUE TO TEMPERATURE EFFECT

13.8.1 One two-hinged arch and one tied arch are shown in Fig. 13.8 depicting the effect of temperature rise on the arch ribs. Due to temperature rise, the arch rib ACB will have an increase in length to AC′B for the two-hinged arch and to AC′B′ for the tied arch. The effect of temperature in case of two-hinged arch will be different from that for tied arches. In case of the former, since there is no displacement of the supports, the

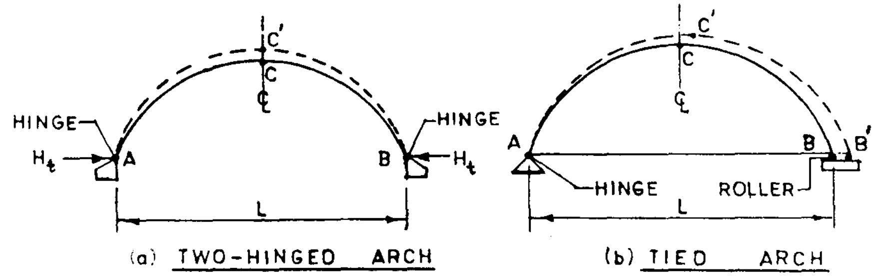


FIG. 13·8 TEMPERATURE EFFECT ON ARCH

increase in length of the arch rib will offer thrust, H_t, on the supports and the crown of the arch will go vertically up from C to C′. In case of the latter, however, the roller will try to allow the free end B to move to B′ and as such will try to release the thrust but the tie, on the other hand, will try to hold the end B in position until it is stretched to such an extent that the tensile force in the tie is equal to the thrust of the arch. This force for tied arches will be less than that for the hinged arches, (span, rise etc. of both the arches remaining the same). However, the strain in the tie being small, the reduction of H_t will not be very significant and as such for all practical purposes, both the tie and the arch rib may be designed for H_t even for tied arches.

13.8.2 If, t, is the rise in temperature and α, is the coefficient of expansion then the arch rib ACB will increase in length to AC′B such that AC′B = ACB (1 + αt). If L is the span of the arch then it can be proved that the support B, if free to move due to temperature effect, will go to B′ horizontally such that BB′ = Lαt. That is, by preventing the movement of B, the horizontal expansion of the arch prevented is Lαt. If H_t is the horizontal thrust due to prevention of the expansion of the arch, the bending moment on an element of the arch at a height y from the springing is given by :

$$M = H_t\, y \qquad (13.35)$$

13.8.3 It is known that the horizontal increase in span δL of an arch due to bending moment is given by

$$\delta L = \int_0^L \frac{My\, ds}{EI}$$

Since δL = Lαt

$$L\alpha t = \int_0^L \frac{My}{EI}\, ds \qquad (13.36)$$

Putting the value of M in equation 13.36 from equation 13.35. We have,

$$L\alpha t = \int_0^L \frac{H_t\, y^2}{EI}\, ds \qquad (13.37)$$

13.8.4 The cross-section and as such the moments of inertia of an arch section varies from maximum at abutments to minimum at crown. For the purpose of design, the moment of inertia of any section x may be taken as $I = I_C \sec\theta$ where I_C is the moment of inertia of crown section and θ is the slope of the arch.

Substituting ds = dx sec θ and I = I_C Sec θ, equation 13.37 becomes —

$$L\alpha t = \int_0^L \frac{H_t\, y^2\, dx}{EI_C} \qquad (13.38)$$

$$\therefore \quad H_t = \frac{L\alpha t}{\frac{1}{EI_C}\int_0^L y^2\, dx} = \frac{EI_C\, L\alpha t}{\int_0^L y^2\, dx} \qquad (13.39)$$

13.8.5 As mentioned in Art. 13.6, shrinkage and plastic flow of concrete shortens the arch rib and as such H becomes a pull on the abutments. Temperature fall will also cause a pull and therefore, the effect of temperature fall shall also be duly considered alongwith shrinkage and plastic flow of concrete to cater for the worst conditions.

13.9 FORCES AND MOMENTS DUE TO ARCH SHORTENING

13.9.1 As explained in Art. 13.6, due to arch shortening, part of the horizontal force caused by external loading is reduced. Horizontal force due to external loading is given by :

$$H = \frac{\int_0^L \frac{M_1\, y\, ds}{EI}}{\int_0^L \frac{y^2\, ds}{EI}} \qquad (13.40)$$

The reduced value of H due to external loading including the effect of arch shortening may be given by the following expression :

$$H_a = \frac{\int_0^L \frac{M_1\, y\, ds}{EI}}{\int_0^L \frac{y^2\, ds}{EI} + \int_0^L \frac{ds}{AE}} \qquad (13.41)$$

Where M_1 = Bending moment at any section due to external loads the arch being considered as simply supported beam.

A = Area of cross-section of the arch rib at any point.

E = Young's Modulus of arch concrete.

When E is constant for the same arch and ds = dx sec θ A = A_C Sec θ (approx.) and I = I_C sec θ, equation 13.41 becomes :

$$H_a = \frac{\int_0 M_1\, y dx}{\int_0^L y^2\, dx + \int_0^L \frac{I_C\, dx}{A_C}} \qquad (13.42)$$

If H_s is known, moment M_s, at any section of the arch due to external loading including the effect of arch shortening can be evaluated from the expression given below :

$$M_s = (M_1 - H_s y) \tag{13.43}$$

13.10 FORCES AND MOMENTS DUE TO SHRINKAGE AND PLASTIC FLOW OF CONCRETE

13.10.1 As outlined in Art. 13.6, the effect of shrinkage of the arch rib is similar to that due to temperature fall. The shrinkage strain, C_S, may, therefore, replace the temperature strain, αt in equation 13.39 to get the pull H_S due to shrinkage.

$$\therefore\ H_S = \frac{EI_C\, LC_S}{\int_0^L y^2\, dx} \tag{13.44}$$

13.10.2 As regards the effect of plastic flow of concrete, the value of E may be modified to half of instantaneous value while determining the forces and moments. On examination of the expressions 13.39, 13.40, 13.42 and 13.44 for the evaluation of the horizontal forces it may be noted that only the temperature and shrinkage are affected by the plastic flow of concrete since the expressions concerning these effects only contain E term.

ILLUSTRATIVE EXAMPLE 13.2

A two-hinged parabolic arch of 40m span is loaded with 120 KN load at each fourth point (Fig. 13.9). The rise of the arch is 5m. The moment of inertia of the arch rib varies as the secant of the slope of the arch. Find the forces and moments considering the effect of temperature variation, arch shortening, shrinkage and plastic flow of concrete.

Given : $\alpha = 11.7 \times 10^{-6}$ *per degree centrigade,* $C_S = 4 \times 10^{-4}$, $E = 31.2 \times 10^4$ Kg/cm², $t = 18°C$, $A_C = bxd = 30 \times 150$ cm $= 4500$ cm², $I_C = 8.5 \times 10^6$ cm⁴.

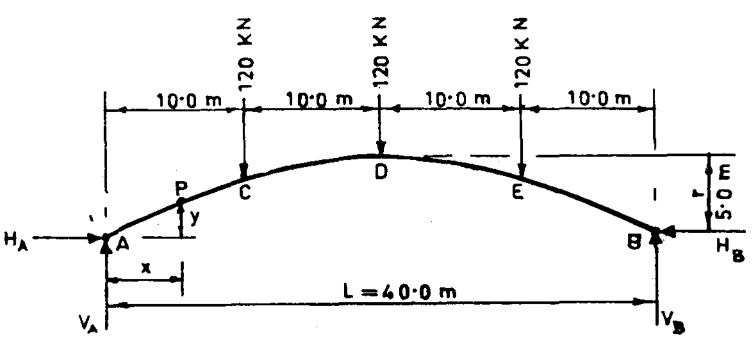


FIG. 13·9 DETAILS OF ARCH (Example 13·2)

Solution

From equation 13.10, the equation of a parabolic arch rib is

$$y = \frac{4rx}{L^2}(L - x) = \frac{4 \times 5x}{(40)^2}(40 - x) = \frac{x}{80}(40 - x) \tag{13.45}$$

Considering the arch as simply supported beam, $V_A = V_B = \frac{3 \times 120}{2} = 180$ KN.

Moment from A to C = 180x KNm

Moment from C to D = 180x – 120(x – 10) = 60x + 1200 KNm

Moment from D to E = 180x – 120(x – 10) – 120(x – 20) = 3600 – 60x KNm

Moment from E to B = 180x – 120(x –10)–120(x – 20) – 120(x – 30) = 7200 – 180x KNm

From equation 13.16, $H = \dfrac{\int_0^L \frac{Myds}{EI}}{\int_0^L \frac{y^2ds}{EI}}$

Since $I = I_C \sec\theta$ and $ds = dx \sec\theta$,

$$H = \frac{\int_0^L Mydx}{\int_0^L y^2dx} \tag{13.46}$$

Integration of the numerator :

(a) *Section A to C*

$$\int Mydx = \int_0^{10} (180x)\frac{x}{80}(40 - x)dx = 24{,}375 \text{ KNm}^3$$ (Putting the value of y from equation 13.45)

(b) *Section C to D*

$$\int Mydx = \int_{10}^{20} (60x + 1200)\frac{x}{80}(40 - x)dx = 96{,}875 \text{ KNm}^3$$

(c) *Section D to E*

$$\int Mydx = \int_{20}^{30} (3600 - 60x)\frac{x}{80}(40 - x)dx = 96{,}875 \text{ KNm}^3$$

(d) *Section E to B*

$$\int Mydx = \int_{30}^{40} (7200 - 180x)\frac{x}{80}(40 - x)dx = 24{,}375 \text{ KNm}^3$$

$$\text{Total} \int^L Mydx = 24{,}375 + 96{,}875 + 96{,}875 + 24{,}375 = 2{,}42{,}500 \text{ KNm}^3$$

Integration of the denominator

$$\int_0^L y^2dx = \int_0^{40}\left[\frac{x}{80}(40-x)\right]^2 dx = 533\ m^3 \qquad \therefore\ H = \frac{\int_0^L Mydx}{\int_0^L y^2dx} = \frac{2{,}42{,}500\ KNm^3}{533\ m^3} = 455\ KN$$

Bending moments for external loads and horizontal thrusts :

$$y \text{ at } C = \frac{x}{80}(40-x) = \frac{10}{80}(40-10) = 3.75\ m; \quad y \text{ at } D = 5.0\ m$$

$\therefore$ Moment at A = Moment at B = 0 (since the arch is hinged at A & B)

Moment at C = Moment at E = $(M - Hy) = (V_A x - Hy) = 180 \times 10 - 455 \times 3.75 = 93.75$ KNm

Moment at D = $V_A x - 120(x - 10) - Hy = 180 \times 20 - 120(20 - 10) - 455 \times 5 = 125$ KNm

Temperature Effect

As stated in Art. 13.6.2, the effective temperature variation is taken as $\frac{2}{3}$ of the actual temperature variation, i.e. $\frac{2}{3} \times 18 =$ 12°C. From equation 13.39, $H_t = \dfrac{EI_C L\alpha t}{\int_0^L y^2dx}$

$$\text{Numerator } = 31.2 \times 10^4 \times 8.5 \times 10^6 \times 40 \times 10^2 \times 11.7 \times 10^{-6} \times 12$$

$$= 1490 \times 10^3\ Kgm^3 = 14.6 \times 10^3\ KNm^3$$

Denominator (as obtained previously in this example) = 533 m^3 $\quad \therefore\ H_t = \dfrac{14.6 \times 10^3}{533} = 27.4$ KN

Arch Shortening

From equation 13.42, the value of H including the effect of arch shortening is given by –

$$H_a = \frac{\int_0^L Mydx}{\int_0^L y^2dx + \int_0^L \frac{I_C dx}{A_c}}$$

As previously worked out in this example, $\int_0^L Mydx = 2{,}42{,}500$ KNm and $\int_0^L y^2dx = 533\ m^3$

$$\int_0^L \frac{I_C dx}{A_c} = \frac{I_C}{A_c}\int_0^L dx = \frac{I_C}{A_c} \times 40\ m. \quad = \frac{8.5 \times 10^6 \times 40 \times 10^2}{4500} = 7.56 \times 10^6\ cm^3 = 7.56\ m^3$$

$$\therefore \text{ Reduced } H_a = \frac{2{,}42{,}500}{533 + 7.56} = 448.6\ KN$$

Effect of Shrinkage :

Coefficient of shrinkage, $C_S = 4 \times 10^{-4}$

If the arch rib is concreted in sections as recommended in Art. 13.6 to reduce shrinkage, this value may be taken as 50 percent of C_S i.e. 2×10^{-4}.

From equation 13.44, $H_S = \dfrac{EI_C L C_S}{\int_0^L y^2 dx}$

Numerator $= 31.2 \times 10^4 \times 8.5 \times 10^6 \times 40 \times 10^2 \times 2 \times 10^{-4} = 212 \times 10^{10}$ Kg cm^3 = 20,800 KNm^3

Denominator as obtained previously = 533 m^3 $\quad \therefore \; H_S = \dfrac{20{,}800}{533} = 39.0$ KN

Effect of Plastic Flow :

As mentioned in Art. 13.10, the value of E may be taken as half while estimating the temperature and shrinkage effect. Therefore, the values of H_t and H_S may be reduced by 50 percent in consideration of the plastic flow of concrete of the arch rib.

Summary of Results :

(a) H due to external loads = 455 KN (Thrust)

(b) H_a considering arch shortening = 448.6 KN (Thrust)

(c) H_t due to temperature including plastic flow = 50% of 27.4 = ±13.7 KN (Thrust or pull)

(d) H_S due to shrinkage including plastic flow = 50% of 39.0 = (–) 19.5 KN (pull)

$\therefore$ Maximum H = 448.6 + 13.7 – 19.5 = 442.8 KN (thrust)

Minimum H = 448.6 – 13.7 – 19.5 = 415.4 KN (thrust)

Design Moment on the arch rib at various section :

$y_C = \dfrac{x}{80}(40 - x) = \dfrac{10}{80}(40 - 10) = 3.75$ m

$M_C = M_E = V_A x - H_A$ (minimum) $y_C = 180 \times 10 - 415.4 \times 3.75 = 1800 - 1558 = 242$ KNm

$M_D = 180 \times 20 - 120 \times 10 - 415.4 \times 5.0 = 3600 - 1200 - 2077 = 2400 - 2077 = 323$ KNm

Bending moments at various sections of the arch are shown in Fig. 13.10. It may be noted that the horizontal thrust induced in the arch rib has reduced the free bending moments by nearly 87 percent.

13.11 NORMAL THRUST ON ANY SECTION OF THE ARCH

13.11.1 For design of any section of the arch rib, the magnitude of bending moment and the normal thrust must be known as explained in Art. 13.1.4. The bending moments for dead loads and other effects such as temperature, arch shortening, shrinkage, plastic flow etc. may be obtained as outlined before. The bending moments for live loads may be obtained by the use of influence lines as explained hereafter in Art. 13.13. Therefore, in order to get all the design forces and moments for each critical section of the arch, not only the bending moments but also the thrusts and shears must be known.

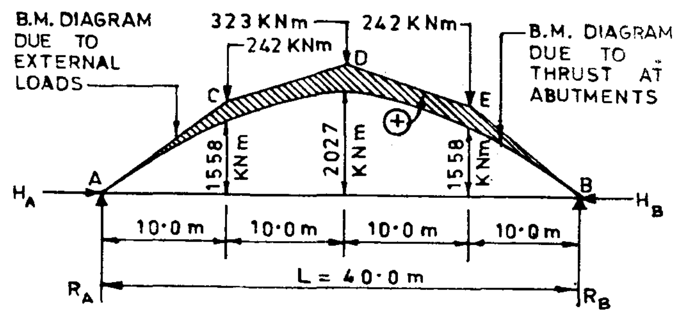


FIG. 13·10 NETT B.M.DIAGRAM (Hatched)(Ex.13·2)

The procedure is now explained. The normal thrust for any section X of the arch rib at a distance x from A and subjected to horizontal thrust, H and vertical thrust, V is given by $P_X = H\cos\theta + V\sin\theta$. If there is a moving load W acting on the arch then the normal thrust at a section X (at a distance x from A) is given by —

(a) *When the load W is within A to X*

$$P_X = H_A\cos\theta + V_A\sin\theta - W\sin\theta$$

$$= H_A\cos\theta - (W - V_A)\sin\theta = H_A\cos\theta - V_B\sin\theta \quad (13.47)$$

(b) *When the load is between X to B*

$$P_X = H_A\cos\theta + V_A\sin\theta \quad (13.48)$$

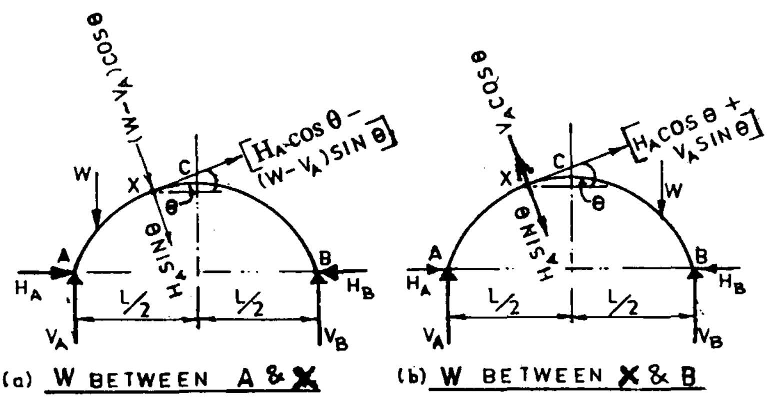


FIG. 13·11 NORMAL THRUST AND RADIAL SHEAR

13.12 RADIAL SHEAR

13.12.1 For the design of any section, values of bending moment, shear and normal thrust shall be known. The method of determination of bending moment and normal thrust has been explained in Art. 13.7 to 13.11. In this article, the evaluation of radial shear is explained.

As in normal thrust, if the moving load W is between A to X, radial shear S_X at a section is given by —

$S_X = H_A \sin\theta - V_A \cos\theta + W \cos\theta = H_A \sin\theta + (W - V_A) \cos\theta$

$$= H_A \sin\theta + V_B \cos\theta \tag{13.49}$$

When the load W is between X to B,

$$S_X = H_A \sin\theta - V_A \cos\theta \tag{13.50}$$

13.13 INFLUENCE LINES

13.13.1 In the previous articles, the procedure for the determination of moments, thrust and shear for any section for static loads was discussed. In case of bridges, the vehicles the bridge has to carry, are not static but movable and therefore, the evaluation of moment, thrust and shear has to be done with the assistance of influence lines as described previously. Method of drawing influence lines for two hinged parabolic arch is indicated in Art. 13.13.2.

13.13.2 Influence Lines for Two-hinged Parabolic Arches

13.13.2.1 ***Influence lines for horizontal thrust at abutments***

From equation 13.18, $H = \dfrac{\int_0^L M_1 y\,dx}{\int_0^L y^2 dx}$

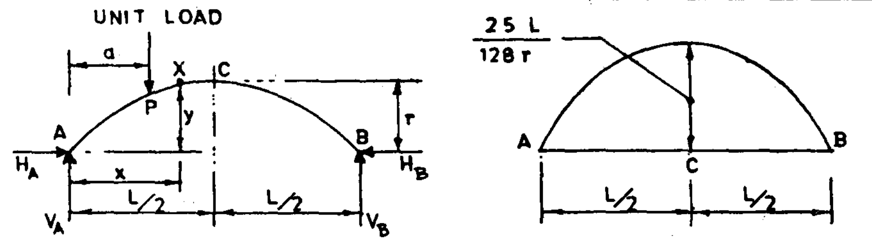
Horizontal thrust in a two-hinged arch carrying a unit concentrated load at P at a distance of 'a' from origin is given by,

$$H = \frac{5a}{8\,rL^3}(L - a)(L^2 + La - a^2) \tag{13.51}$$

H is maximum when the unit load is at the crown, C where, $a = \frac{L}{2}$.

$$\therefore \quad H = \frac{5 \times \frac{L}{2}}{8\,rL^3}\left(L - \frac{L}{2}\right)\left[L^2 + \frac{L^2}{2} - \frac{L^2}{4}\right] = \frac{25\,L}{128\,r} = 0.1953\,\frac{L}{r}$$

The complete influence line diagram for thrust, H is shown in Fig. 13.12b. The co-efficient for ordinates of the influence line diagram for various values of 'a' are given in Table 13.1.



(a) UNIT LOAD AT P AND SECTION X UNDER CONSIDERATION

(b) INFLUENCE LINE DIAG. FOR THRUST, H

FIG. 13·12 THE ARCH & THE INFLUENCE LINE DIAG. FOR, H

TABLE 13.1 : CO-EFFICIENTS FOR ORDINATES OF INFLUENCE LINE DIAGRAM FOR THRUST, H OF A TWO-HINGED PARABOLIC ARCH *(Scott)*[5]

Position of unit load at $\frac{a}{L}$	0	0.1	0.2	0.25	0.3	0.4	0.5
Co-efficient	0	0.0613	0.1160	0.1392	0.1588	0.1860	0.1953

Note :

(a) The ordinates for the I.L. diagram = coefficient $\times \frac{L}{r}$.

(b) The thrust due to a concentrated load W = ordinate × W.

(c) The thrust due to a distributed loads, ω/m = Area of inf. line diag × ω.

13.13.2.2 *Influence line diagram for Bending Moment at a section X*

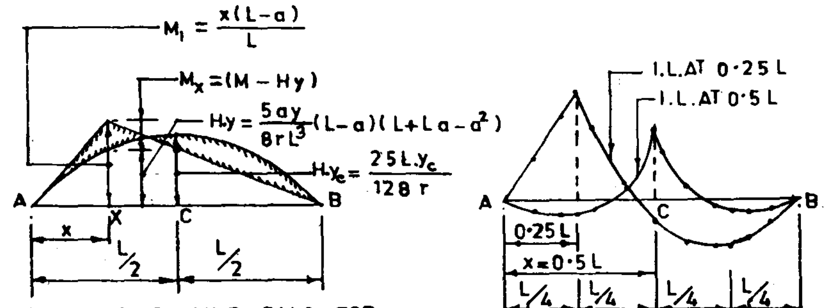
Bending moment at X for a unit load at P is given by $M_X = M_1 - Hy$.

The ordinate of M_1 at X is $\frac{x(L-a)}{L}$ and the free M_1 diagram is a triangle with height equal to $\frac{x(L-a)}{L}$. When the unit load is at 'a', the thrust, H is equal to $\frac{5a}{8rL^3}(L-a)(L^2+La-a^2)$ as given in equation 13.51. Therefore, hogging moment at X due to H will be Hy, i.e. $\frac{5ay}{8rL^3}(L-a)(L^2+La-a^2)$

Hence moment at X due to a unit load at P will be $M_X = M_1 - Hy$

$$= \frac{x(L-a)}{L} - \frac{5ay}{8rL^3}(L-a)(L^2+La-a^2) \qquad (13.52)$$

The influence line diagram for moment at X (generalised diagram) is shown is Fig. 13.13a and the same at x = 0.25L and x = 0.5L (i.e. at crown) are shown in Fig. 13.13b, the co-efficients for ordinates for moments at various sections (i.e. x = 0, 0.1L, 0.2L etc.) for various load position (i.e. a = 0, 0.1L, 0.2L etc.) are shown in Table 13.2. The ordinates for the influence line diagram shall be obtained by multiplying the coefficients with L. The moment M_X for a concentrated load W = coefficient × WL.



(a) INFLUENCE LINE DIAG. FOR Mx (Generalised diagram)

(b) INFLUENCE LINE DIAG. FOR Mx AT X= 0·25 L AND X = 0·5 L

FIG. 13·13

TABLE 13.2 : CO-EFFICIENTS FOR MOMENT ORDINATES FOR INFLUENCE LINE DIAGRAM OF A TWO-HINGED PARABOLIC ARCH *(Scott)*[5]

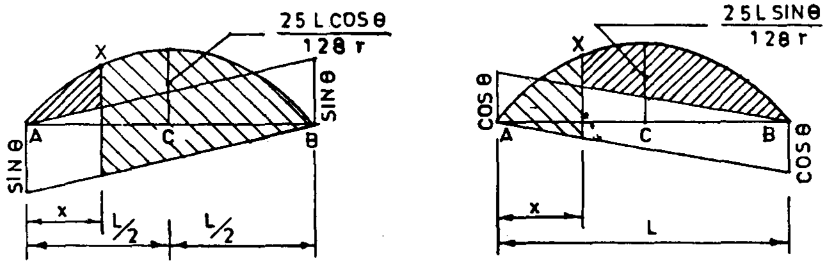
Position of Section at $\frac{x}{L}$	Position of unit load at $\frac{a}{L}$												
	0	0.1	0.2	0.25	0.3	0.4	0.5	0.6	0.7	0.75	0.8	0.9	1.0
0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.1	0	0.068	0.038	0.025	0.013	–0.007	–0.020	–0.027	–0.027	–0.025	–0.022	–0.012	0
0.2	0	0.041	0.086	0.061	0.038	0.001	–0.025	–0.039	–0.042	–0.039	–0.034	–0.019	0
0.25	0	0.029	0.063	0.083	0.056	0.010	–0.022	–0.040	–0.044	–0.042	–0.037	–0.021	0
0.3	0	0.018	0.043	0.058	0.077	0.024	–0.014	–0.036	–0.043	–0.042	–0.037	–0.022	0
0.4	0	0.001	0.009	0.016	0.028	0.061	0.013	–0.019	–0.033	–0.034	–0.031	–0.019	0
0.5	0	–0.011	–0.016	–0.014	–0.009	0.014	0.055	0.014	–0.009	–0.014	–0.016	–0.011	0

Note :
a) The ordinate for IL diagram = Coefficient × L
b) The moment M_x due to a concentrated load, W = ordinate × W
c) The moment M_x due to a distributed load, ω/m = Area of inf. line diag. × ω

13.13.2.3 *Influence Line Diagram for Normal Thrust at Section X*

Normal thrust at any section X is obtained by using equation 13.47 or 13.48 i.e. $P_X = H_A \cos\theta - V_B \sin\theta$ or $H_A \cos\theta + V_A \sin\theta$ depending on whether the load is at the left or right of section X respectively.

The influence lines for $V_A \sin\theta$ and $V_B \sin\theta$ are two parallel lines having end ordinates equal to $\sin\theta$ since V_A or V_B for unit moving load at ends becomes unity. The influence line for $H \cos\theta$ is $\cos\theta$ times the influence line for H as obtained previously. The influence line diagram for P_X is shown in Fig. 13.14a.



(a) INFLUENCE LINE FOR NORMAL THRUST AT SECTION, X

(b) INFLUENCE LINE FOR RADIAL SHEAR AT SECTION, X

FIG. 13·14 (Khurmi)[3]

13.13.2.4 *Influence Line Diagram for Radial Shear at X*

Radial shear at X is given by the equation $S_X = H_A \sin\theta + V_B \cos\theta$ or $H_A \sin\theta - V_A \cos\theta$ depending on whether the unit load is on the left or on the right of section X vide Art. 13.12.

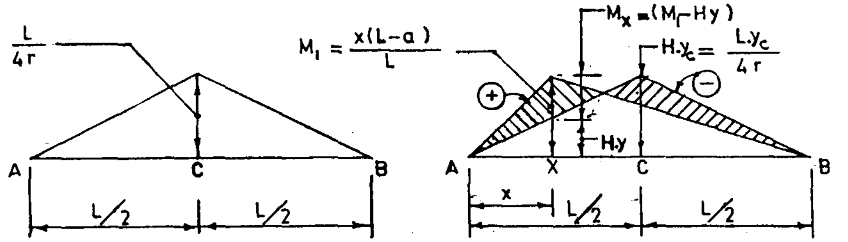
The influence lines for $V_A \cos\theta$ and $V_B \cos\theta$ are two parallel lines having end ordinates equal to $\cos\theta$ with unit moving load. The influence line for $H \sin\theta$ is $\sin\theta$ times the influence line for H as obtained previously. The final influence line diagram for radial shear at X is shown in Fig. 13.14b.

13.13.3 Influence Line Diagram for Three-hinged Arches and Fixed Arches

13.13.3.1 The influence line diagrams for thrusts on abutments, moments, normal thrusts and radial shear at a section X for three hinged arches and fixed arches may be drawn in the same way as explained in the case of two-hinged arches. However, for ready reference, the influence line diagrams for horizontal thrust, H and for moment at section X for a three-hinged parabolic arch are shown in Fig. 13.15 and those for a fixed parabolic arch are shown in Fig. 13.16. Influence line diagrams for moments at sections x = 0.2L and x = 0.4L for three-hinged arch and at sections x = 0.2L and x = 0.5L for fixed parabolic arches are shown in Fig. 13.17a and 13.17b respectively. The coefficients for ordinates for thrust, H and moments at various sections both for three-hinged and fixed parabolic arches are given in Table 13.3, 13.4, 13.5 and 13.6.

TABLE 13.3 : CO-EFFICIENTS FOR ORDINATES OF INFLUENCE LINE DIAGRAM FOR THRUST, H, OF A THREE-HINGED PARABOLIC ARCH *(Scott)*[5]

Position of unit load at $\frac{a}{L}$	0	0.1	0.2	0.25	0.3	0.4	0.5
Co-efficient	0	0.05	0.10	0.125	0.15	0.20	0.25



(a) I.L. DIAG. FOR HORZONTAL THRUST, H

(b) I.L. DIAG. FOR M_x AT SEC., X (GENERALISED DIAGRAM)

FIG. 13·15 INFLUENCE LINE DIAGRAM FOR THREE HINGED PARABOLIC ARCHES (Scott)[5]

Note : (a) The ordinate for influence line diagram = coefficient $\times \frac{L}{r}$.

(b) The thrust due to a concentrated load, W = ordinate × W.

(c) The thrust due to a distributed load, ω/m = Area of Inf. L. diag × ω.

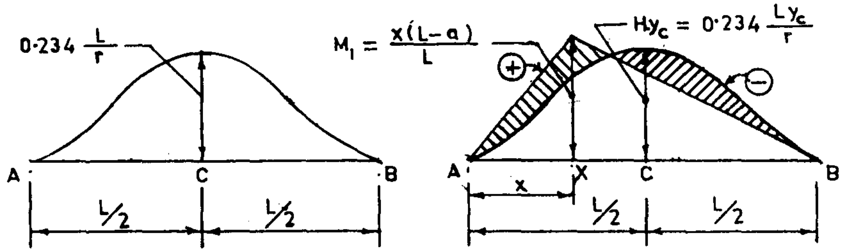
TABLE 13.4 : CO-EFFICIENTS FOR ORDINATES OF INFLUENCE LINE DIAGRAM FOR THRUST, H OF A FIXED PARABOLIC ARCH *(Scott)*[5]

Position of unit load at $\frac{a}{L}$	0	0.1	0.2	0.25	0.3	0.4	0.5
Co-efficient	0	0.030	0.096	0.130	0.165	0.216	0.234

Note : (a) The ordinate of I.L. diagram = coefficient $\times \frac{L}{r}$.

(b) The thrust, H for a point load, W = co–eff. $\times \frac{WL}{r}$ = ordinate × W.

(c) The thrust, H for a distributed load, ω/m = Area of influence line diag x ω.



(a) I.L. DIAG. FOR HORIZONTAL THRUST, H

(b) I. L. DIAG. FOR M_x AT SEC., X (GENERALISED DIAGRAM)

FIG. 13·16 INFLUENCE LINE DIAGRAM FOR FIXED PARABOLIC ARCHES (Scott)[5]

TABLE 13.5 : CO-EFFICIENTS FOR MOMENT ORDINATES FOR INFLUENCE LINE DIAGRAM OF A THREE-HINGED PARABOLIC ARCH *(Scott)*[5]

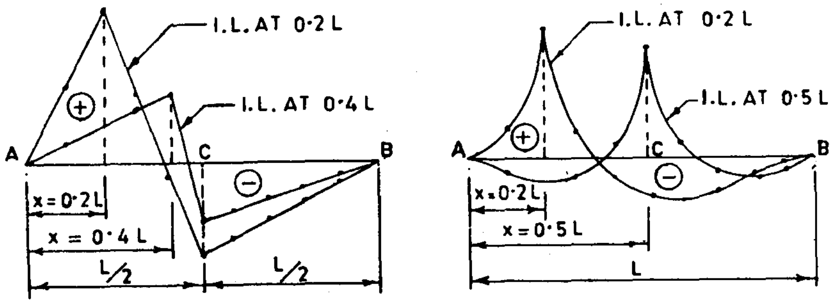
Position of Section at $\frac{x}{L}$	Position of unit load at $\frac{a}{L}$												
	0	0.1	0.2	0.25	0.3	0.4	0.5	0.6	0.7	0.75	0.8	0.9	1.0
0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.1	0	0.072	0.044	0.030	0.016	–0.012	–0.040	–0.032	–0.024	–0.020	–0.016	–0.008	0
0.2	0	0.048	0.096	0.090	0.044	–0.008	–0.060	–0.048	–0.036	–0.030	–0.024	–0.012	0
0.25	0	0.037	0.075	0.094	0.062	0	–0.062	–0.050	–0.037	–0.031	–0.025	–0.012	0
0.3	0	0.028	0.056	0.070	0.084	0.012	–0.060	–0.048	–0.036	–0.030	–0.024	–0.012	0
0.4	0	0.012	0.024	0.030	0.036	0.048	–0.040	–0.032	–0.024	–0.020	–0.016	–0.008	0
0.5	0	0	0	0	0	0	0	0	0	0	0	0	0

Note :
a) The ordinate of I.L. diagram = coefficient δ L
b) The moment Mx for a concentrated load W = Ordinate × W
c The moment Mx for a distributed load ω/m = Area of influence line diag. × ω

TABLE 13.6 : CO-EFFICIENTS FOR MOMENT ORDINATES FOR INFLUENCE LINE DIAGRAM OF A FIXED PARABOLIC ARCH *(Scott)*[5]

Position of Section at $\frac{x}{L}$	Position of unit load at $\frac{a}{L}$										
	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
0	0	–0.061	–0.064	–0.037	0	0.031	0.048	0.047	0.032	0.011	0
0.1	0	0.026	–0.009	–0.018	–0.013	–0.003	0.006	0.009	0.008	0.003	0
0.2	0	0.014	0.054	0.014	–0.009	–0.019	–0.020	–0.015	–0.009	–0.003	0
0.3	0	0.005	0.024	0.060	0.013	–0.016	–0.028	–0.027	–0.017	–0.006	0
0.4	0	–0.001	0.002	0.018	0.052	0.006	–0.019	–0.025	–0.019	–0.007	0
0.5	0	–0.005	–0.012	–0.010	0.008	0.047	0.008	–0.010	–0.012	–0.005	0

Note :
a) The ordinate for I.L. diagram = coefficient × L
b) The moment M_X due to a concentrated load W = ordinate × W
c) The moment M_X due to a distributed load, ω/m = Area of inf. line diag. × ω



(a) I.L. DIAG. FOR M_x FOR THREE HINGED PARABOLIC ARCHES

(b) I.L. DIAG. FOR M_x FOR FIXED PARABOLIC ARCHES

FIG. 13·17 (Scott)[5]

13.13.3.2 *The use of Influence Line coefficients in the evaluation of Thrust and Moments with Static Loads*

As mentioned in Art. 13.13.1, the influence line diagrams are used for the evaluation of maximum horizontal thrust, moment etc. for moving loads. These influence line diagrams and tables may also be used for the determination of thrust, moment etc. for any static load also. This is illustrated in the illustrative example 13.3.

ILLUSTRATIVE EXAMPLE 13.3

Evaluate the thrust and moments for the parabolic arch as given is Illustrative Example13.2 and Fig. 13.9, by the use of influence line diagrams and coefficients.

Solution

From Table 13.1, the coefficients for thrust for unit load at 0.25L, 0.5L and 0.75L are 0.1392, 0.1953 and 0.1392 respectively.

Therefore, thrust $H = \Sigma \text{ coefficient} \times \frac{WL}{r}$

$$= 0.1392 \times (120 + 120) \times \frac{40.0}{5.0} + 0.1953 \times 120 \times \frac{40.0}{5.0} = 267.26 + 187.49 = 454.75 \text{ KN}$$

Thrust as determined previously = 455 KN. Hence the value obtained by the use of influence line coefficients agrees with the previous value calculated by the use of formulae.

Again from Table 13.2, the coefficients for moments at C (x=0.25L), D (x=0.5L) and E (x=0.75L) for loads at C (a=0.25L), D (a=0.5L) and E (a=0.75L) are as below :

Coefficients at C or E (i.e. at 0.25L or 0.75L)

(a) Unit load at 0.25L	=	0.083
(b) Unit load at 0.5L	=	–0.022
(c) Unit load at 0.75L	=	–0.042

Coefficients at D (i.e.at 0.5L)

(a) Unit load at 0.25L	=	–0.014
(b) Unit load at 0.5L	=	0.055
(c) Unit load at 0.75L	=	–0.014

∴ Moment at C or E = Σ coefficient × WL = (0.083 – 0.022 – 0.042) × 120 × 40.0

= 91.2 KNm (Previous value = 93.75 KNm).

Moment at D = Σ coefficient × WL = (–0.014 + 0.055 – 0.014) × 120 × 40.0

= 129.6 KNM (Previous value = 125 KNm)

Therefore, the values obtained by the use of influence line coefficient agree with those by using formula. The small variation is due to the approximate coefficients (upto three places of decimals) used in the table. Though approximate, the method by the use of influence line coefficients is very quick and as such this has some advantage over the previously used method.

13.14 DESIGN PROCEDURE

(1) Select the type of arch to be adopted; fix up span, rise of arch etc.

(2) Assume rough section of the arch rib and find the thrust and bending moment at different sections for various dead loads such as deck structure, wearing course, columns and beams etc.

(3) Draw influence line diagrams for various sections for moments and thrust and determine the live load moments and thrust due to live loads.

(4) Compute the moments and thrust due to temperature variation, shrinkage, rib shortening etc.

(5) Tabulate the positive moments and thrusts and also negative moments and thrusts for different sections due to various design and loading conditions and find the design moments and thrusts.

(6) Evaluate the normal thrusts and radial shears at critical sections both for dead and live loads.

(7) Check the sections for concrete and steel stresses. If found satisfactory, detailing of reinforcement may be taken up; if not, the previous procedures are to be repeated, where necessary, with revised trial section of the arch.

13.15 HINGES FOR CONCRETE ARCHES

13.15.1 The hinges are capable of transmitting thrust, pull or shear but cannot resist bending moments. Therefore, sometimes in the construction of arch bridges, the bending stresses induced by shrinkage, rib shortening (due to dead load only), settlement of centering, settlement of the abutments etc. which are of temporary nature may be eliminated by providing temporary hinges at the crown and at the springings. These temporary hinges do away with the moments at the critical sections viz. crown and springings.

13.15.2 After the construction is over, the gap in the hinges is filled with well graded and well compacted concrete so that the section is able to resist bending moments, thrusts that may be induced by the subsequent loads such as balance dead load, live load, temperature, residual shrinkage and rib shortening due to live load etc. One form of temporary hinge is illustrated in Fig. 13.18.

13.15.3 Permanent hinges provided in arch bridges should be strong enough to sustain thrust, shear etc. due to combined loads during service of the bridge. These hinges will not offer any resistance to moments and therefore, these locations will be points of zero moments.

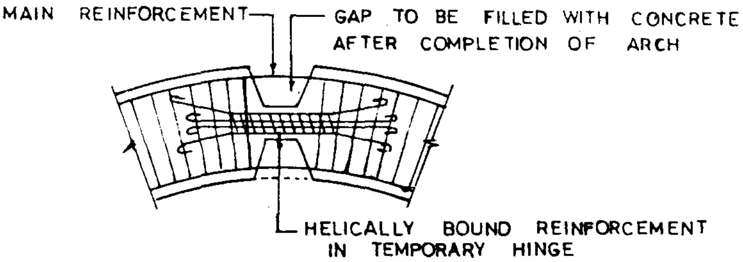
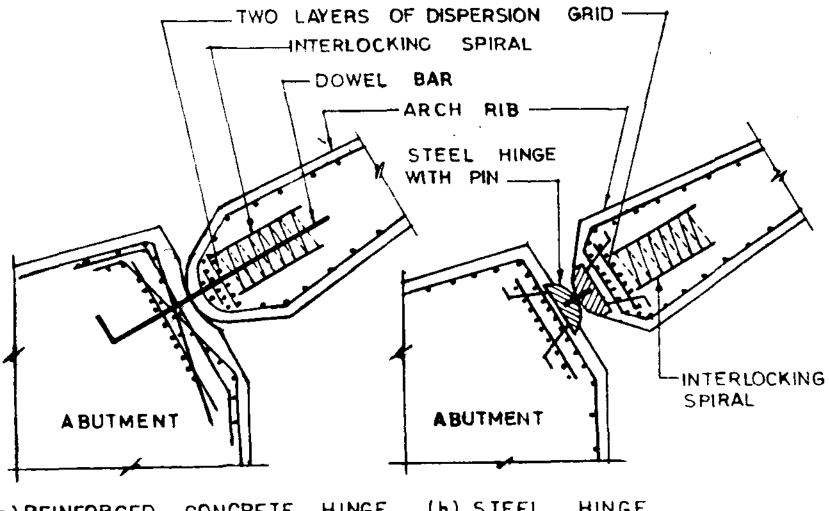


FIG. 13·18 TEMPORARY HINGE AT CROWN (Scott)[5]

Fig. 13.19 shows one steel and one concrete permanent hinge. Curvature in these hinges is very important and as such proper curvature should be maintained. The curvature in steel hinges is made during casting and finishing. The curvature in concrete hinges may be achieved by screeding the concave sunface with a wooden screed and placing a soft wood over the concave surface so as to form the convex surface. Instead of using the soft wood, plaster of paris may also be employed over the screeded concave surface to form the convex surface.



(a) REINFORCED CONCRETE HINGE (b) STEEL HINGE

FIG. 13·19 PERMANENT HINGES AT SPRINGING (Scott)[5]

13.16 ABUTMENTS

13.16.1 Abutments for arch bridges are usually made of mass concrete so as to get large dead weight due to which it may be possible to make the thrust from the arch axis more vertical. The base section of the abutments is made in such a way that the resultant thrust under all conditions of loading passes through as near the centre of the base as possible. When founding the abutments on rock, necessary benching should be done on rock for better stability.

13.16.2 Sometimes, cellular type R.C. abutments are made to effect economy in cost. To get the necessary dead weight of the abutments, the inside of the cellular portion is filled with earth. This helps in making the thrust more inclined towards vertical axis. The thrust from the arch rib is transmitted through the counterforts to the base raft. The counterforts should, therefore, be strong enough to sustain the thrust coming on them. Both these types of abutment are illustrated in Fig. 13.20.

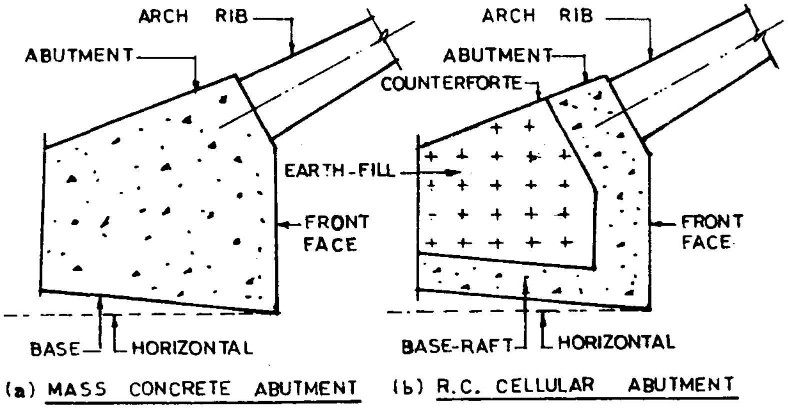


FIG. 13·20 ABUTMENTS FOR ARCH BRIDGES

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CHAPTER 14

SIMPLY SUPPORTED STEEL BRIDGES

14.1 GENERAL

14.1.1 Long span steel bridges such as continuous plate girder bridges, continuous box-girder bridges, continuous truss bridges, arch bridges etc. will be discussed in Chapter 17. In this chapter, simply supported short to medium long span steel bridges of non-composite section are described. These are :—

i) Rolled Steel beam bridges.
ii) Plated beam bridges.
iii) Plate girder bridges.
iv) Trussed girder bridges.

14.1.2 The merits and demerits of steel bridges are as follows :

Merits :

i) The bridges can be constructed quickly.
ii) Steel being homogeneous material, performance is better.
iii) The beams being produced in a factory, quality is ensured.

Demerits :

i) Cost is higher than concrete bridges.
ii) Requires regular maintenance.
iii) Maintenance cost is high.
iv) Unsuitable for corrosive atmosphere.

14.1.3 The main components of a steel bridge are :

i) Main girders
ii) Flooring
iii) Bracings.
iv) Bearings.

14.2 ROLLED STEEL BEAM BRIDGES

14.2.1 This is the simpliest type steel bridge having RSJ as the girder and steel trough plate filled with concrete or reinforced concrete slab as the bridge deck as shown in Fig. 14.1.

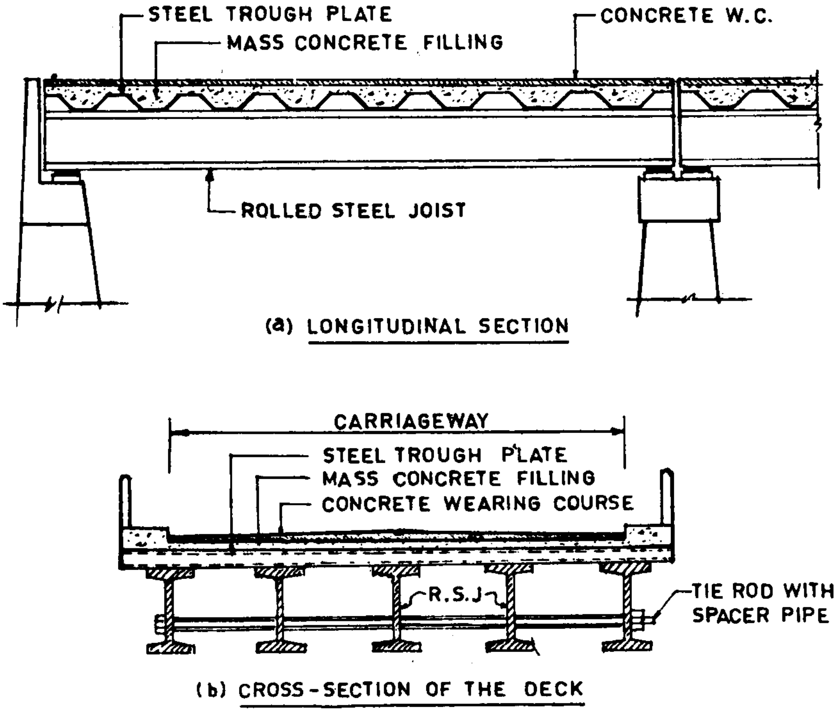

FIG. 14.1 STEEL TROUGH PLATE WITH CONCRETE DECKING

14.2.2 These bridges have very small spans and are constructed over canals or small channels where scour is negligible and shallow foundations are possible to reduce foundation cost. Since the load carrying capacity of these bridges is limited, these bridges are suitable for village roads where both the laden weight and frequency of the vehicular traffic are less.

14.3 PLATED BEAM BRIDGES

14.3.1 Plated beam bridges can cover comparatively larger spans than the RSJ bridges since their section modulus is increased by increasing the flange areas with additional plates fixed to the flanges by rivetting or welding (Fig. 14.2).

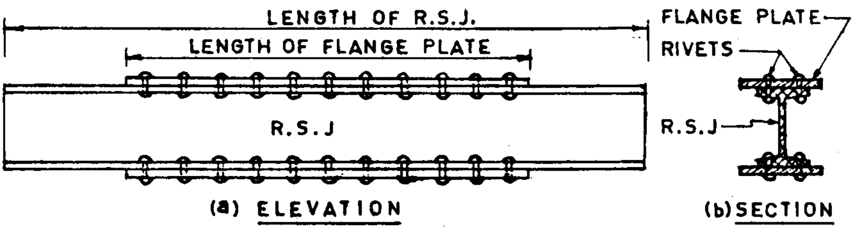

FIG. 14.2 PLATED BEAM

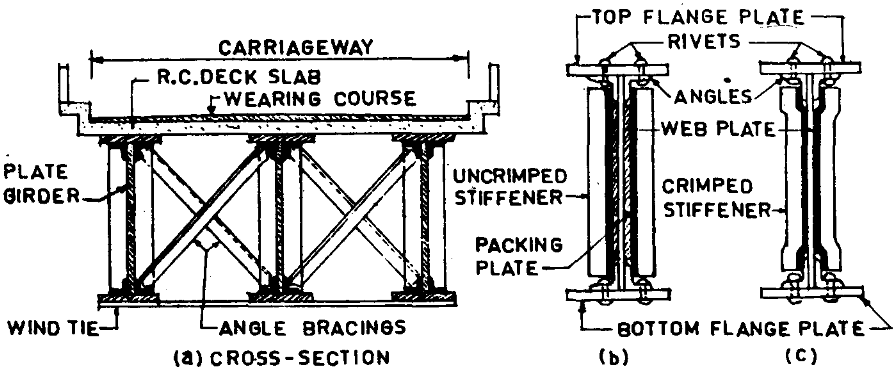

FIG. 14.3 DETAILS OF PLATE GIRDER BRIDGE

14.4 PLATE GIRDER BRIDGES

14.4.1 When the span of the bridge is beyond the spanning capacity of plated beam bridges, plate girder bridges are adopted. In such bridges, the depth of the girder from bending and deflection consideration is such that rolled steel joists are not suitable and therefore, the girders are fabricated with plates and angles either by rivetting or by welding.

14.4.2 If the bridge is through type then only two girders can be used one on either side but in case of deck type bridges, any number of girders can be used depending upon the economic consideration.

14.4.3 The section modulus required for the plate girder at various sections such as mid-section, one-third section, one fourth sections etc. varies depending upon the moment at these sections and as such the flange plates may be curtailed at the point of less moments such as at the ends for simply supported girders.

14.4.4 The components of a plate girder are as given below (Fig. 14.4) :

1. Web plate
2. Flange plates
3. Flange angles
4. Rivets or welds connecting flange angles with the flange plates and web plate.
5. Vertical stiffeners fixed to the web plate at intervals along the length of the girder to guard against buckling of web plate.

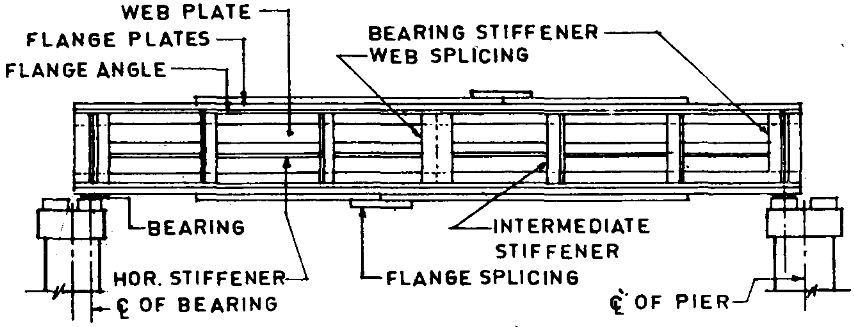

FIG. 14.4 COMPONENTS OF A PLATE GIRDER

6. Horizontal stiffeners fixed to the web plate depth wise, one or more in numbers, to prevent buckling of web plate.
7. Bearing stiffeners at the ends over the centre line of bearing and at intermediate points under the point loads.
8. Web splice-plates used to join the two web plates.
9. Flange splice-plates used to join the two flange plates.
10. Angle splice-plates used to join the two flange angles.
11. Bearing plates at the ends resting on the piers/abutments.

14.4.5 Full length of plates and angles for the fabrication of the plate girder may not be available for which splicing is necessary. The flange plates are normally spliced near the ends for simply supported spans while the web plate is spliced at or near the centre.

14.4.6 To guard against buckling of the web plate, vertical and horizontal stiffeners are provided by use of m.s. angles. At each end and also at the point of concentrated heavy loads, bearing stiffeners are necessary for transmission of loads. The bearing stiffeners are uncrimped and packing plate is used in between the web and the stiffening angle but intermediate angle stiffeners are usually crimped.

14.4.7 The design of a plate girder involves the following steps :

1. Computation of B.M. and S.F. at various sections say one-fourth, one-third and one-half span.
2. Estimation of required section moduli at various sections.
3. Design of web from shear consideration.
4. Design of flange angles and flange plates to obtain the required section modulii at various sections.
5. Curtailment of flange plates and flange angles in consideration of reduced values of required section modulii near the end sections.
6. Design of rivets or welds connecting various members such as flange angles with web plate and flange angles with flange plates.
7. Design of splices such as flange splice and web splice.
8. Design of stiffeners.
9. Design of bearing plates.

ILLUSTRATIVE EXAMPLE 14.1

A simply supported plate girder bridge of 20 metres span carries a dead load of 50 KN/m excluding self weight of the girder and also a live load of 60 KN/m per girder. Design the plate girder at the centre of the span considering the impact allowance as per IRC code.

Solution

Dead load = 50 KN/m.

$$\text{Impact factor} = \frac{9}{13.5 + L} = \frac{9}{13.5 + 20} = 0.269;$$

Live load with impact = $60 \times 1.269 = 76.14$ KN/m. Total superimposed load with impact excluding self weight of girder = $50 + 76.14 = 126.14$ KN/m.

Self weight of plate girder per metre length is approximately given by $\frac{WL}{300}$, where W is the total superimposed load per metre and L is the span in m.

$\therefore$ Self weight of plate girder $= \frac{WL}{300} = \frac{126.14 \times 20}{300} = 8.41$ KN/m

Total superimposed load plus self weight of girder = 126.14 + 8.41 = 134.55 KN/m. say 135 KN/m.

Max. bending moment $= \frac{wL^2}{8} = \frac{135 \times 20^2}{8} = 6750$ KNm. Max. shear force $= \frac{wL}{2} = \frac{135 \times 20}{2} = 1350$ KN

Design of web plate

Assume thickness of web plate, t_w = 12 mm. Economical depth of a plate girder is given by

$$d = 1.1\sqrt{\frac{M}{f_b\, t_w}}$$

Where, M = Maximum bending moment; f_b = Allowable bending stress; t_w = Thickness of web plate.

$$\therefore \quad d = 1.1\sqrt{\frac{6750 \times 10^6}{138 \times 12}} = 2018 \text{ mm}$$

Adopt depth of web = 2000 mm.

Design of flange plates

Net flange area required for tension flange, $A_t = \frac{M}{f_b d} = \frac{6750 \times 10^6}{138 \times 2000} = 24{,}456$ mm^2. If 4 Nos. 22 mm. dia rivets are used for connecting flange plates to flange angles and 4 Nos. rivets for connecting flange angles to web plate and if 2 nos. 500 mm × 16 mm. flange plates and 2 nos. 200 mm × 100 mm × 15 mm flange angles are used to fabricate the plate girder then the net flange area available are as follows :

Gross area of flange plates	2 × 500 × 16	16,000 mm^2
Deduction for rivet holes	4 × 23.5 × 16	1,504 mm^2
	Net flange plate area	14,496 mm^2
Gross area of flange angles	2 × 4278	8,556 mm^2
Deduction for rivet holes	4 × 23.5 × 15	1,410 mm^2
	Net flange angle area	7,146 mm^2

Net equivalent web area taken as flange area $= \frac{\text{Area of web}}{8} = \frac{2000 \times 12}{8} = 3000$ mm^2

$\therefore$ Total net flange area = 14,496 + 7,146 + 3,000 = 24,642 mm^2

Net flange area required = 24,456 mm^2. Hence safe.

Compression flange area required for resisting compression $= \frac{M}{f_b d} = \frac{6750 \times 10^6}{138 \times 2000} = 24{,}456$ mm^2

Compression flange area provided = 16,000 + 8,556 + 3,000 = 27,556 mm^2. (Gross area ; no deduction for holes)

The details of the plate girder is shown in Fig. 14.5.

Check for bending stress :

Moment of inertia of the section

$$= \frac{1}{12} \times 12 \times 2000^3 + 4 \times 4278 \times (1000 - 22.2)^2 + 2 \times 2 \times 500 \times 16 \times 1016^2 = 57{,}393 \times 10^6 \text{ mm}^4$$

$\therefore$ Section modulus, $Z = \frac{I}{Y} = \frac{57{,}393 \times 10^6}{1016} = 56.49 \times 10^6 \text{ mm}^3$

$\therefore$ Stress in tension flange $= \frac{M}{Z} = \frac{6750 \times 10^6}{56.49 \times 10^6} = 119.49 \text{ N/mm}^2$

Allowable stress 138 N/mm^2

Check for shear stress

Shear stress $\frac{V}{bd} = \frac{1350 \times 10^3}{12 \times 2000} = 56.25 \text{N/mm}^2$

Allowable shear stress = 83.3 N/mm^2. Hence safe.

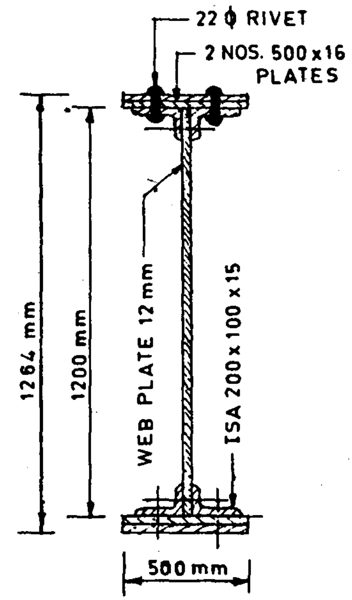

FIG. 14·5 (Ex. 14·1)

14.5 TRUSSED GIRDER BRIDGES

14.5.1 Trussed girder or truss bridges have an upper or top chord, lower or bottom chord and web members which are vertical and diagonals. For a simply supported truss bridge, the upper chord is subjected to compression and the lower chord is subjected to tension. The web members may be only diagonals as in Warren Truss (Fig. 14.6a) or a combination of verticals and diagonals as in modified Warren Truss (Fig. 14.6b) or Pratt Truss (Fig. 14.6c & 14.6d) or Howe Truss (Fig. 14.6e) or Parker Truss (Fig. 14.6g). For larger spans, the panels are again subdivided from structural considerations as in truss with diamond bracing (Fig. 14.6f), Pettit Truss (Fig. 14.6h) or K-truss (Fig. 14.6i). The span range for a simply supported truss bridge is 100 to 150 metres. For larger spans, truss bridges shall be either continuous or cantilever bridges which are discussed in Chapter 17.

14.5.2 The truss bridges may be either of deck type or of through type (Fig: 14.7) i.e. the bridge deck will be near the top chord in the former type and near the bottom chord in the latter type. It is, therefore, needless to say that parallel chord trusses which are shown in Fig. 14.6a to 14.6e may be either of deck type or through type as in Fig. 14.7a and 14.7b but trusses with curved top chord as shown in Fig. 14.6g to 14.6i are invariably of through type (Fig. 14.7c).

14.5.3 The bridge deck is on longitudinal girders resting on cross-girders which transfer the loads to the trusses at each panel joints. Details of a truss bridge are shown in Fig. 14.8. Since no load comes on the truss members except at panel joints, the truss members are subjected to direct stress only, either tensile or compressive, and no bending moment or shear force occurs in the truss members. The panel joints where members meet are assumed as hinged and therefore, no bending moment in the truss members is developed even due to the deflection of the truss.

14.5.4 Determination of Forces in Statically Determinate Trusses

The forces in the truss members are determined by the following methods when the trusses are statically determinate.

1. Graphical Method by Stress or Force Diagrams.

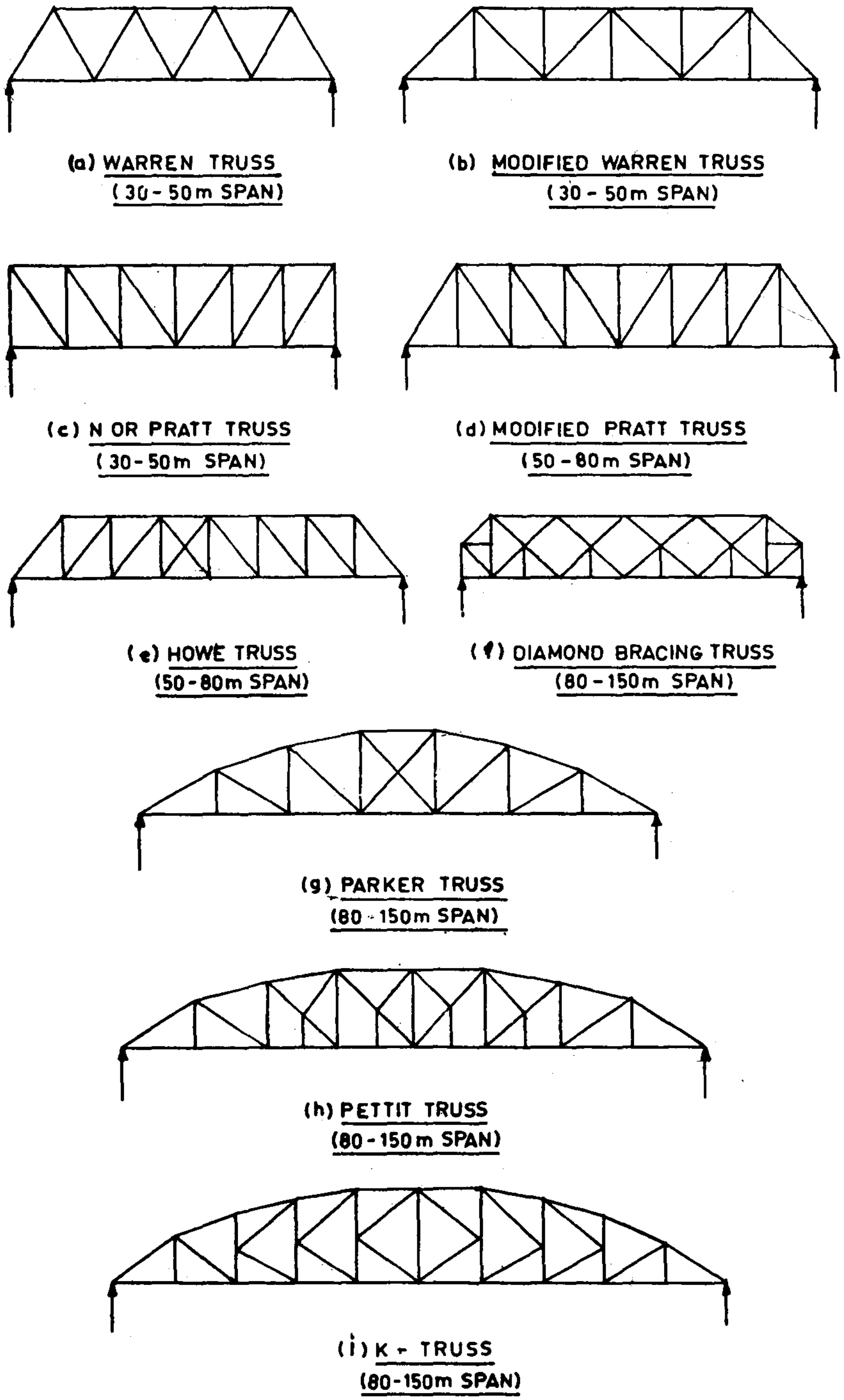

FIG.14·6 VARIOUS TYPES OF BRIDGE TRUSSES

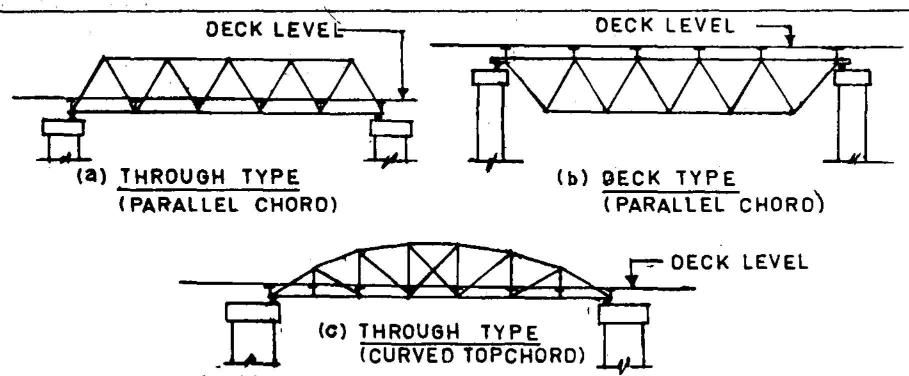

FIG.14.7 THROUGH OR DECK TYPE BRIDGE

2. Method of Sections.
3. Method of Resolutions.

The above methods are explained by one illustrative example.

ILLUSTRATIVE EXAMPLE 14.2

A simple equilateral triangular truss with a load of 30 KN at joint 2 of the truss is shown in Fig. 14.9a. Calculate the forces in the members of the truss by the above mentioned three methods, one by one.

Graphical Method

The members are numbered with 0 at the centre of the truss and A, B, C at the outside and counted clockwise.

Therefore, reactions are AB and CA. The members are OB, OC and OA. Reaction AB = Reaction CA = 15 KN. Since the loads and reactions are vertical, a force diagram in a suitable scale is drawn (Fig. 14.9b) which

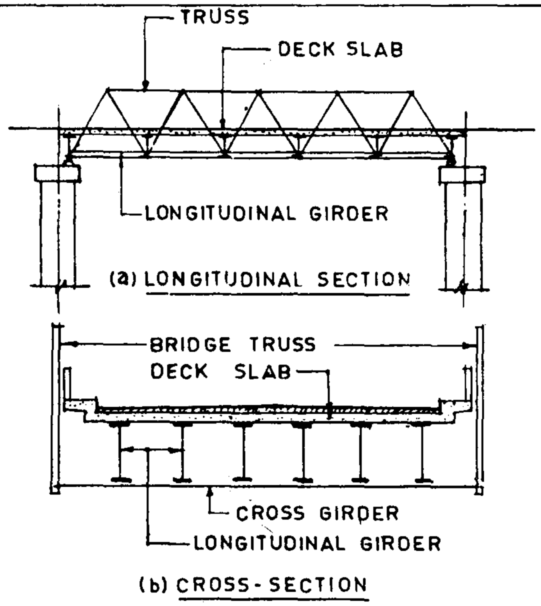

FIG. 14.8 DETAILS OF TRUSS BRIDGE

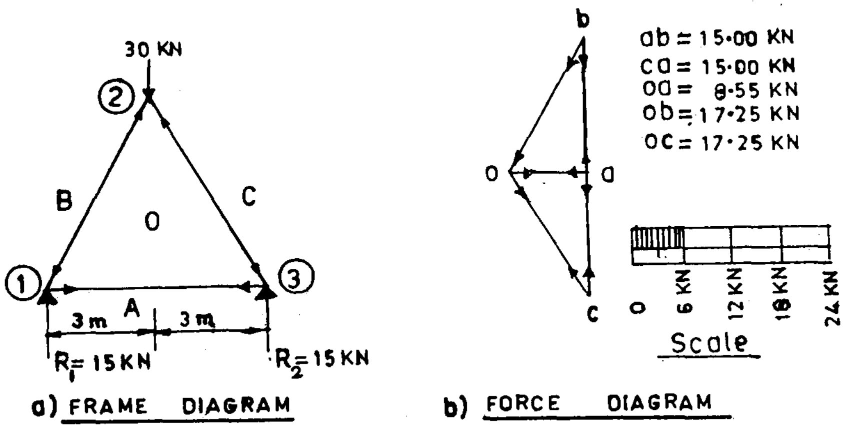

FIG. 14·9 (Ex.14·2)

is also vertical. In this diagram, *bc* downwards represents W, *ca* upwards represents R_2 and *ab* upwards represents R_1. Since $R_1 + R_2 = 30$ KN, in the force diagram also $bc = ca + ab = 15 + 15 = 30$ KN.

Now the force diagram is drawn. Considering the joint 1 of the frame, a line, *bo*, is drawn on the force diagram parallel to BO and a line, *ao*, is drawn on the force diagram parallel to AO. The triangle, *oab*, is the triangle of force diagram for the joint 1 and *ab*, *bo*, *oa*, represent to scale the reaction R_1 and internal forces in BO, OA respectively. Similarly in joint 2, W is the external load or force represented by, *bc*, in the force diagram. Lines *ob* and oc are drawn parallel to member OB and OC. The triangle, *bco*, is the triangle of force diagram for the joint 2 and *bc*, *co*, *ob* represent to scale the reaction W, and internal forces in OC & OB respectively. The triangle of force diagram for joint 3 viz. *cao*, is similarly drawn ; ca, ao and oc representing to scale the reaction R_2 and internal forces in the member AO and OC respectively.

The values of the internal forces in the members are known from the force diagram as illustrated above. The nature of the force viz. whether the force is tensile or compressive can also be determined from the same force diagram. In any triangle of force diagram, the path of the forces starting from the known force is followed in the same direction and these directions are indicated in the frame diagram. For example, in the triangle of force diagram *a b o*, *ab* (= reaction R_1) is known to act upwards. Following this path, the direction of force *bo* and *oa* will be as shown in the force diagram and is also shown in the frame diagram. A force towards a joint in the frame diagram indicates a compressive force and a force away from the joint is a tensile force. Thus, in joint 1, the known force is $ab = R_1$ acting upwards and following this path, the directions of forces for *bo* and *oa* in the force diagram and for member BO and OA in the frame diagram are shown. The direction of force BO is towards the joint and therefore, is a compressive force. Similarly, the direction of force OA is away from the joint and is therefore, a tensile force. In the same way and starting from the force whose direction is known, the directions of all the forces are shown in the frame diagram and thus the nature of all the forces are known.

Method of Sections

In this method, the member whose force is to be determined is cut by a line which also cuts some other members of the frame. Start shall be made from a point where only one force is unknown. The frame will remain balanced even by the cut if external forces act in the cut members as shown in Fig. 14.10 in the same simple frame as in Fig. 14.9.

The forces may be determined by taking moment about a convenient joint so that only one known and one unknown forces are involved. For example in Fig. 14.10b, a cut X-X is made in the frame cutting member AO and BO. Taking moment about joint 2, $f_{OA} \times \frac{\sqrt{3}}{2} \times 6 = 15 \times 3$ or, $f_{OA} = 8.66$ KN i.e. away from the joint and hence tensile force.

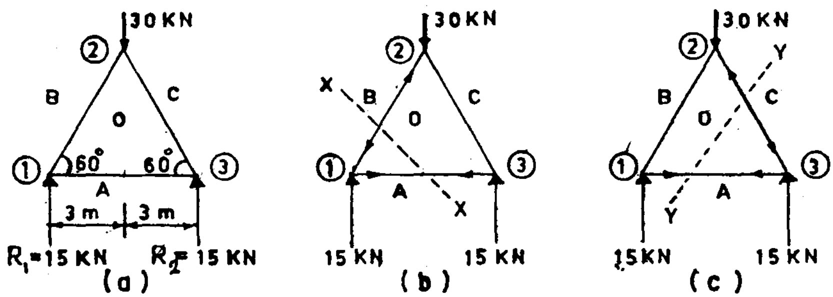

FIG. 14·10 (Ex.14·2)

Taking moment about joint 3, $f_{OB} \times \frac{\sqrt{3}}{2} \times 6 = 15 \times 3$ $\therefore$ f_{OB}=17.32 KN i.e. towards the joint, i.e. compressive force.

Similarly, the force f_{OC} may be known by a cut Y-Y and taking moment about joint 1.

Therefore, the forces in the members determined by the method of sections are as below :

$f_{OB} = f_{OC} = 17.32$ KN (Compressive), $f_{OA} = 8.63$ KN (tensile)

Method of Resolutions

In this method, all the forces and the external loads at a joint are resolved in horizontal and vertical direction and equated to zero since the joint is in equilibrium. Start shall be made from the joint where external load is acting and no more than two unknowns are there. The same numerical example as shown in Fig. 15.9 is taken to illustrate this method also. The force towards a joint is compressive and the force away from the joint is tensile.

Considering joint 1 and resolving f_{OB} in the horizontal and vertical direction and equating to zero, $f_{OB} \sin 60° + 15 = 0$ or $f_{OB} = (-)\frac{15 \times 2}{\sqrt{3}} = (-)\ 17.32$ KN, i.e. compressive and $f_{OB} \cos 60° + f_{OA} = 0$ or $f_{OA} = (-)\ f_{OB} \cos 60° = (-)\ 17.32 \times \frac{1}{2} = (-)\ 8.66$ KN i.e. tensile.

Considering joint 3, $f_{OC} \cos 60° + f_{OA} = 0$ or or $f_{OC} = (-)\ 8.66 \times 2 = (-)\ 17.32$ KN compressive.

The forces in the frame as obtained by Method of Resolutions are : $f_{OB} = f_{OC} = 17.32$ KN compressive. $f_{OA} = 8.66$ KN tensile.

Therefore, it may be noted that the forces in the frame are the same as worked out by the Method of Sections and the Method of Resolution. The values as worked out by the Graphical Method slightly differ as they are to be scaled and as such error in measurement occurs. However, for all practical purposes, these values are acceptable and the design may be proceeded with without any hesitation.

14.5.5 Determination of Forces in Trusses with one Redundant Member

14.5.5.1 When, however, the trusses are statically indeterminate, the forces in the member can not be determined by the methods as described in Art. 14.5.4. Therefore, some other methods are to be applied in finding out the forces in such trusses, two of which are discussed below :

1. Method based on Principle of Least Work.
2. Maxwell's Method.

14.5.5.2 ***Method Based on Principle of Least Work***

A corollary of Castigliano's theorem is that the work done in stressinga structure under a given system of loads is the least possible consistent with the maintenance of equilibrium. Therefore, the differential coefficient of the work done with respect to one of the forces in the structure is equal to zero. This is the *"Principle of Least Work* which is utilised in evaluating the forces in statically indeterminate trusses.

The strain energy stored or work done in any member of length, L and cross sectional area, A, under a direct force, P, is given by

$$u = \frac{P^2 L}{2\,AE} \tag{14.1}$$

And the work done in the whole structure is

$$U = \sum \frac{P^2 L}{2\,AE} \tag{14.2}$$

In evaluating the forces in the truss member, the procedure is as follows :

1. Remove the redundant member and calculate the forces in the remaining members of the truss (which is now statically determinate) due to external loading. The forces in the members due to above are F_1, F_2, F_3 (say).
2. Remove the external loading and apply a unit pull in the redundant member and find out the forces in the truss members.
3. If K_1, K_2, K_3 etc. are the forces in the members due to unit pull in the redundant member and if the actual force in the redundant member of the truss due to external loading is T then the total force in the members will be, T for the redundant member (since F = 0) and $(F_1 + K_1T), (F_2 + K_2T), (F_3 + K_3T)$ etc. for other members.
4. Total work done in the structure including that in the redundant member will be

$$U = \frac{T^2 L_o}{2\,A_oE} + \sum \frac{(F + KT)^2 L}{2\,AE} \tag{14.3}$$

5. The differential coefficient of the work done with respect to the force T in the redundant member is therefore given by —

$$\therefore \frac{\partial U}{\partial T} = \frac{2\,TL_o}{2\,A_oE} + \sum \frac{2\,(F + KT)\,KL}{2\,AE} = \frac{TL_o}{A_oE} + \sum \frac{(F + KT)\,KL}{AE}$$

The principle of least work states that $\frac{\partial U}{\partial T} = 0 \quad \therefore \frac{TL_o}{A_oE} + \sum \frac{(F + KT)\,KL}{AE} = 0$

$$\therefore \; T = (-)\frac{\sum \frac{FKL}{AE}}{\left(\frac{L_o}{A_oE} + \sum \frac{K^2 L}{AE}\right)} \tag{14.4}$$

Since the work done in the redundant member is taken into consideration separately vide equation 14.3, the summation of the terms $\sum \frac{FKL}{AE}$ & $\sum \frac{K^2 L}{AE}$ shall be for the remaining members only.

14.5.5.3 *Maxwell's Method*

This method is also based on the total work done in stressing the structure but the basic difference of this method with the previous one is that instead of inducing an internal force T, in the redundant member, this force is applied as an external load. This means that in the previous method based on Principle of Least Work, the strain energy of the redundant member is also included in the total work done since the force T in the redundant member is an internal one but in Maxwell's method, the force T is an external one and therefore, does not contribute towards the total work done due to stressing of the structure. In Maxwell's Method, the first theorem of Castigliano is utilised in evaluating the forces in the redundant member as described below :

1. Step 1 to step 4 same as in the previous method. However, in step 3, unit load and T are external loads along the redundant member.
2. Total work done excluding that in the redundant member will be

$$U = \sum \frac{(F + KT)^2 L}{2\,AE} \tag{14.5}$$

3. $$\frac{\partial U}{\partial T} = \sum \frac{2(F + KT)\,KL}{2\,AE} = \sum \frac{(F + KT)\,KL}{AE}$$

   As per Castigliano's first theorem, the differential coefficient of the total strain energy in a structure with respect to any load gives the deformation of the structure along the direction of the load. Therefore, $\frac{\partial U}{\partial T}$ gives the deformation of the redundant member in the direction T.

$$\therefore \sum \frac{(F + KT)\,KL}{AE} = \delta \tag{14.6}$$

4. But as a result of the force T in the redundant member, the deformation of the member is also given by the following relationship :

$$\delta = (-)\,\frac{TL_o}{A_o\,E} \tag{14.7}$$

   Where L_o and A_o are the length and area of cross section of the redundant member.

Minus sign in equation 14.7 is used as the deformation in equation 14.6 gives the value of δ in the direction of T but as a result of the pull, T, the deformation in the member will be in the opposite direction.

Equating δ from equations 14.6 and 14.7, $\sum \frac{(F + KT)\,KL}{AE} = (-)\frac{TL_o}{A_o\,E}$

$$\text{or, } T\left[\frac{L_o}{A_oE} + \sum \frac{K^2 L}{AE}\right] = (-)\sum \frac{FKL}{AE}$$

$$\text{or, } T = (-)\,\frac{\sum \frac{FKL}{AE}}{\left(\frac{L_o}{A_oE} + \sum \frac{K^2L}{AE}\right)} \tag{14.8}$$

The values of T can be determined from equation 14.8 since all other values except T are known. Knowing the value of T, the forces in all the members of the truss can be determined such as T in the redundant member and $(F_1 + K_1T)$, $(F_2 + K_2T)$, $(F_3 + K_3T)$ etc. in other members. It may also be noted that although the truss with redundant member is analysed by two different methods, the result is the same as may be seen from equations 14.4 and 14.8.

ILLUSTRATIVE EXAMPLE 14.3

A bridge truss with a redundant member at the central panel and with 200 KN vertical and 100 KN horizontal loads acting at one of the top panel nodes is shown in Fig. 14.11. Find the forces in all the members of the

truss. The truss is hinged at one support and has roller bearing at the other support. It may be assumed for the convenience of computation that the ratio of length to the cross-sectional area for all the members is the same.

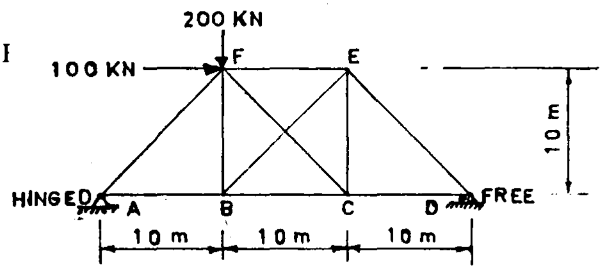

FIG. 14·11 (Ex.14·3) (Khurmi)[7]

Solution by Method of Least Work

1. The redundant member BE is removed and the forces ine all the remaining members of the truss which is now statically determinate is determined by any of the methods as explained in Art. 14.5.3. This is tabulated in Table 14.1. Fig. 14.12a shows external loads and reactions.
2. The external loads are removed, a unit pull is applied in the redundant member (Fig. 14.12b) and the forces, K_1, K_2, K_3 etc. in various members are found. This is shown also in Table 14.1.

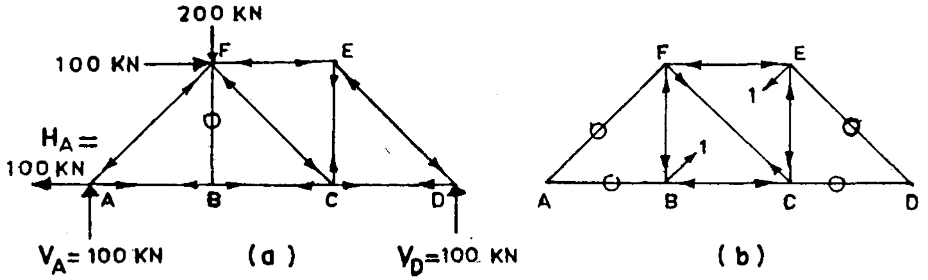

FIG. 14·12 (Ex.14·3) (Khurmi)[7]

TABLE 14.1 : FORCES IN TRUSS MEMBERS IN KN (TENSION +ve; COMPRESSION -ve)

Member	F	K	FK	K^2	KT	Final Force (F + KT)
AB	+200	0	0	0	0	+200
BC	+200	$-\frac{1}{\sqrt{2}}$	$-\frac{200}{\sqrt{2}}$	$+\frac{1}{2}$	−50	+150
CD	+100	0	0	0	0	+100
DE	$-100\sqrt{2}$	0	0	0	0	−141.4
EF	−100	$-\frac{1}{\sqrt{2}}$	$+\frac{100}{\sqrt{2}}$	$+\frac{1}{2}$	−50	−150
FA	$-100\sqrt{2}$	0	0	0	0	−141.4
BF	0	$-\frac{1}{\sqrt{2}}$	0	$+\frac{1}{2}$	−50	−50
CF	$-100\sqrt{2}$	+1	$-100\sqrt{2}$	+1	70.7	−70.7

Contd.

TABLE 14.1 : FORCES IN TRUSS MEMBERS IN KN (TENSION +ve; COMPRESSION -ve)

Member	F	K	FK	K^2	KT	Final Force (F + KT)
CE	+100	$-\frac{1}{\sqrt{2}}$	$-\frac{100}{\sqrt{2}}$	$+\frac{1}{2}$	−50	+ 50
		Total $\sum$	$-\frac{400}{\sqrt{2}}$	+ 3		

From equation 14.4 or 14.8 $T = (-)\dfrac{\sum \frac{FKL}{AE}}{\left(\frac{L_0}{A_0E} + \sum \frac{K^2L}{AE}\right)}$

Given $\frac{L}{A} = \frac{L_0}{A_0}$ = constant. Also E is the same for all members. $\therefore\ T = (-)\dfrac{\Sigma FK}{(1 + \Sigma K^2)}$

Substituting the values of Σ FK & ΣK^2 from Table 14.1,

$T = (-)\dfrac{-\frac{400}{\sqrt{2}}}{(1+3)} = \dfrac{400}{4\sqrt{2}} = +\dfrac{100}{\sqrt{2}} = +70.7$ KN (tensile) i.e. force in the redundant member BE is 70.7 KN (tensile) and forces in other members are obtained in Table 14.1.

14.5.6 Determination of Forces in Trusses with Two or More Redundant Members

The procedure for determining the forces in truss with two or more redundant members are the same with some modification due to presence of more than one redundant member and the Principle of Least Work may also be utilised in this case. This is explained below :

1. Remove the redundant members such that the truss becomes perfect and does not get distorted after the removal of the redundant members. The truss in Fig. 14.13a has two redundant member BG and DG which are removed as shown in Fig. 14.13b. This latter truss is statically determinate and the forces in the members with the external loads are determined. The forces in the members are say F_1, F_2, F_3 etc.
2. Remove the external loading and apply a unit pull in the redundant member BG (Fig. 14.13c). If K_1, K_2, K_3 etc. are the forces in the members due to unit pull in the redundant member BG and if the actual force in the redundant member BG is T due to external loading, then the total forces in the other members will be $(F_1 + K_1T)$, $(F_2 + K_2T)$ etc.
3. Next apply a unit pull in the redundant member DG (Fig. 14.13d), if K'_1, K'_2, K'_3 etc. are the forces in the members due to a unit pull in the redundant member DG and if the actual force in the redundant member DG is T' due to external loading then the forces in the other members will be K'_1T', K'_2T' etc. due to force T' in the redundant member DG.
4. Actual forces in the other members due to step 1 to 3 are $(F_1 + K_1T + K'_1T')$, $(F_2 + K_2T + K'_2T')$ etc.
5. Total work done in the structure including that in the redundant members will be,

Fig. 14.13

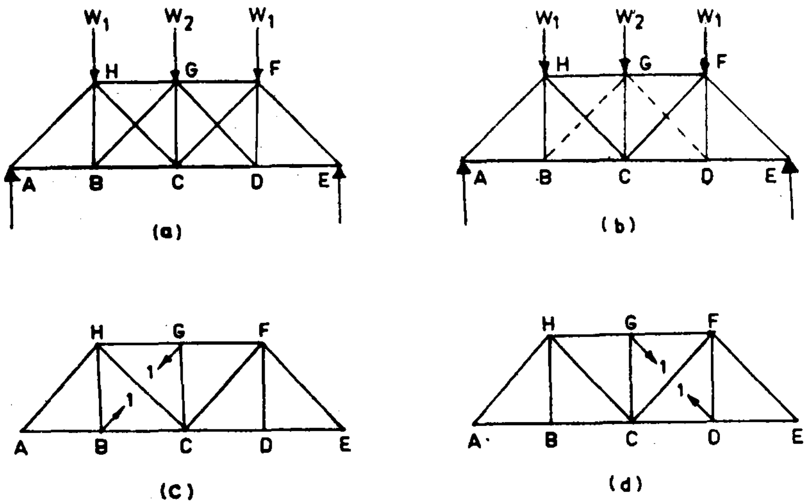

FIG. 14'13 (Khurmi)[7]

$$U = \frac{T^2 L_o}{2 A_o E} + \frac{(T')^2 L'_o}{2 A'_o E} + \sum \frac{(F + KT + K'T')^2 L}{2AE} \qquad (14.9)$$

6. The differential coefficients of the total work done with respect to forces T and T′ are,

$$\frac{\partial U}{\partial T} = \frac{2 TL_o}{2 A_o E} + \sum \frac{2(F + KT + K'T') KL}{2AE}$$

$$= \frac{TL_o}{A_o E} + \sum \frac{(F + KT + K'T') KL}{AE} \qquad (14.10)$$

$$\text{and } \frac{\partial U}{\partial T'} = \frac{T' L'_o}{A'_o E} + \sum \frac{(F + KT + K'T') K'L}{AE} \qquad (14.11)$$

The principle of least work states that,

$$\frac{\partial U}{\partial T} = 0 \text{ and } \frac{\partial U}{\partial T'} = 0 \qquad (14.12)$$

Hence from equation 14.10, 14.11, 14.12,

$$\frac{T L_o}{A_o E} = (-) \left[\sum \frac{FKL}{AE} + \sum \frac{TK^2 L}{AE} + \sum \frac{T' K K' L}{AE} \right] \qquad (14.13)$$

$$\text{and } \frac{T' L_o}{A'_o E} = (-) \left[\sum \frac{FK'L}{AE} + \sum \frac{TK K' L}{AE} + \sum \frac{T'(K')^2 L}{AE} \right] \qquad (14.14)$$

All terms in equation 14.13 and 14.14 are known except T and T′ and as such by solving these two simultaneous equations the values of T and T′ can be computed. By knowing the values of T and T′, the forces in other members are determined from step 4, i.e. $(F_1 + K_1T + K'_1T')$, $(F_2 + K_2T + K'_2T')$ etc. as done in the Illustrative Example 14.3.

14.5.7 Influence Lines for Trussed Bridges

It has been explained in Art. 14.5.4 to 14.5.6 how the forces in the truss members can be determined for static loads. But bridge trusses are subjected to moving loads and as such the forces in the truss members can not be evaluated unless the assistance of the influence lines is taken. Therefore, it is essential to draw the influence lines for forces in the various truss members and the maximum value for each truss member is thus determined after placing the moving loads for maximum effect.

As explained in Art. 14.5.3, the moving loads from the roadway comes on each truss on either side of the roadway at panel joints only. Total load is shared by each truss equally. The influence line diagram for the top and bottom chords are drawn for the BM whereas the influence lines for the diagonal and vertical members are drawn for the S. F. The types of bridge trusses usually used are shown in Fig. 14.6 and the influence lines will vary depending upon the type of truss and location of the member in the truss. However, the principle of drawing the influence line is explained for a parallel chord Pratt truss by an illustrative example.

ILLUSTRATIVE EXAMPLE 14.4

Draw the influence lines for force in the bottom chord AB, top chord LK, diagonals AL & LC and vertical BL of the Pratt truss bridge shown in Fig. 14.14. Also calculate the maximum force in diagonal AL and bottom chord AB if single lane of IRC class AA load crosses the bridge. Panel length = 6m and height of truss = 8m.

Influence Line for Force in Diagonal, AL

Cut bottom chord AB and diagonal AL by a section line 1-1 as shown in Fig. 14.15a. Draw a perpendicular line BN from B on AL. When a unit load moves from one end of the bridge to the other, let the reactions at A and G are R_1 and R_2 respectively. The left portion of the cut truss will be in equilibrium for any position of the unit load in the bridge deck.

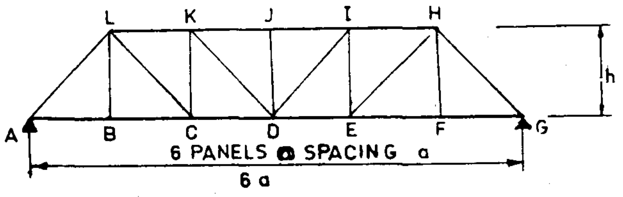

FIG. 14·14 (Example 14·4)

Taking moment about B, $f_{AL} \times BN = R_1 a$ or, $f_{AL} \times \dfrac{ah}{\sqrt{a^2+h^2}} = R_1 a$ or, $f_{AL} = R_1 a \dfrac{\sqrt{a^2+h^2}}{ah}$ (Compression)

But $(R_1\, a)$ is the moment of the freely supported truss girder at B. $\therefore\ f_{AL} = \dfrac{M_B \sqrt{a^2+h^2}}{ah}$ (Compression)

Therefore, the influence line for force in the diagonal AL is equal to $\dfrac{1}{ah}\sqrt{a^2+h^2}$ times the influence line for M_B which is a triangle with ordinate $\dfrac{x\,(L-x)}{L}$ at B, i.e., $\dfrac{a\,(6a-a)}{6a} = \dfrac{5}{6}$ a. Therefore, the ordinate of the influence line for f_{AL} at B will be $\dfrac{5a}{6ah}\sqrt{a^2+h^2} = \dfrac{5}{6h}\sqrt{a^2+h^2}$ as shown in Fig. 14.15b.

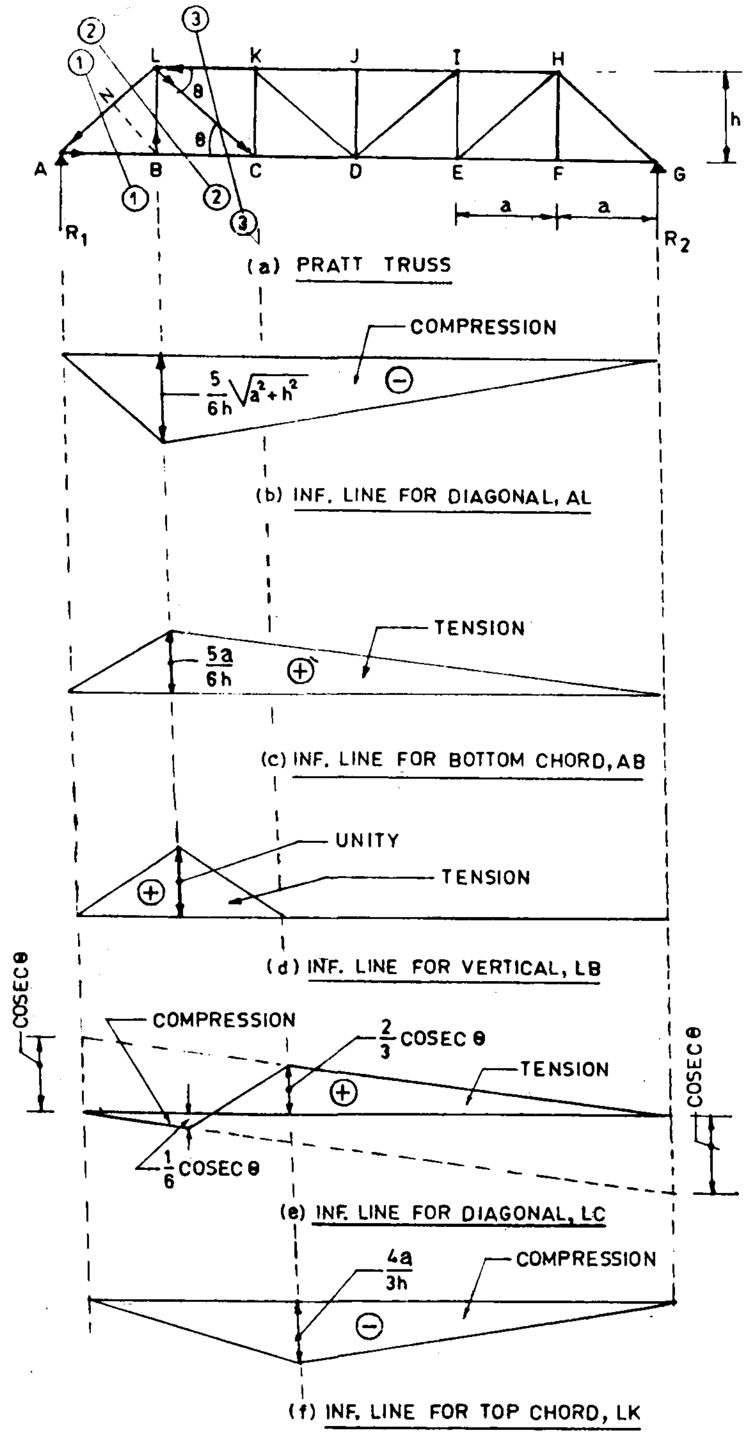

FIG 14·15 INFLUENCE LINES FOR A PRATT TRUSS (Khurmi)[7]

Influence Line for Bottom Chord AB

Consider Section line 1-1 same as before.

Taking moment about L, $f_{AB} \times h = R_1 a$ or, $f_{AB} = \frac{R_1 a}{h} = \frac{M_L}{h}$ (Tension)

Therefore, the influence line for force in the bottom chord AB is equal to $\frac{1}{h}$ times the influence line for M_L which is a triangle with ordinate equal to $\frac{x(L-x)}{L}$ i.e. $\frac{5a}{6}$. Therefore, the ordinate of the influence line for f_{AB} at L is equal to $\frac{5a}{6} \times \frac{1}{h} = \frac{5a}{6h}$ as shown in Fig. 14.15c.

Influence Line for Vertical BL

When a unit load moves from A to B, the tension in the vertical member BL becomes from zero to unity. Again, the tension in BL decreases from unity to zero as the unit load moves from B to C. Thereafter, the tension in BL is always zero when the unit load moves from C to G. Therefore, the influence line for vertical member BL is a triangle having maximum ordinate equal to unity as shown in Fig. 14.15d.

Influence Line for Diagonal LC

Consider cut line 3-3 and that the unit load is moving from A to B. In such case if the equilibrium of the right of cut line 3-3 is considered, it is found that the force in the diagonal LC near the joint C will be downwards since the external force i.e., the reaction R_2 to be balanced by the force in LC is upwards. Therefore, the force in LC will be compressive and its magnitude is given by, $f_{LC} \sin\theta = R_2$ or, $f_{LC} = \frac{R_2}{\sin\theta} = R_2 \operatorname{cosec}\theta$ (Compression).

Next the equilibrium of the truss left of cut line 3-3 is considered when the unit load moves from C to G. Arguing as before, the force in LC near the joint L will be downwards since reaction R_1 acts upwards. Therefore, diagonal LC will be in tension and the magnitude is given by, $f_{LC} \sin\theta = R_1$ or, $f_{LC} = R_1 \operatorname{cosec}\theta$ (Tension)

The influence line for R_1 and R_2 are triangles having ordinates unity and zero at A and G respectively for R_1 and having ordinates zero and unity at A and G respectively for R_2. Therefore, the influence line for LC will be cosec θ times the influence line for R_2 from A to B and compressive in nature. The influence line for LC will be cosec θ times the influence line for R_1 from C to G and tensile in nature. The influence line for LC between B to C will be a line joining the ordinates at B & C which are $\frac{1}{6}\operatorname{cosec}\theta$ (compressive) and $\frac{2}{3}\operatorname{cosec}\theta$ (tensile) respectively. The influence line for LC is shown in Fig. 14.5c.

Influence Line for Top Chord LK

Consider the truss left of cut line 3-3. Taking moment about C, $f_{LK} \times h = R_1 \times 2a$ or, $f_{LK} = \frac{1}{h} \times 2aR_1$ (Compression). But $2aR_1$ is the moment of the freely supported truss at C. $\therefore$ $f_{LK} = \frac{M_C}{h}$ (Compression).

Therefore, the influence line for f_{LK} is $\frac{1}{h}$ times the influence line for M_C the ordinate of which is $\frac{x(L-x)}{L}$ i.e., $\frac{2a(6a-2a)}{6a} = \frac{4a}{3}$ at C. Hence, the ordinate of the influence line for f_{LK} at C is $\frac{1}{h} \times \frac{4a}{3} = \frac{4a}{3h}$

Maximum Forces in Members due to Movement of IRC Class AA Loading

Length of truss = 6a = 6 × 6 = 36 m

Height of truss = h = 8m.

Total load on each truss = 35 tonnes

Length of loading = 3.6 m.

Load intensity per metre length = 9.72 tonnes.

Distribution factor due to eccentricity of loading = 1.2 (say)

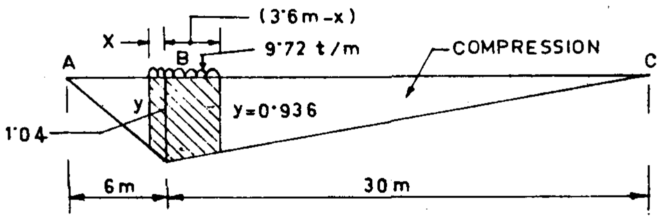

FIG. 14'16 (Ex.14·4)

Impact factor = 10 percent.

Force in Diagonal AL

Ordinate at B of the influence line diagram, $= \frac{5}{6h}\sqrt{a^2+h^2} = \frac{5}{6\times8}\sqrt{6^2+8^2} = \frac{5\times10}{6\times8} = 1.04$

For maximum effect, y for each triangle shall be equal. For left triangle, $y = \frac{1.04\,(6-x)}{6}$

Similarly for right triangle, $y = \frac{1.04\,(26.4+x)}{30}$. $\therefore \frac{1.04\,(6-x)}{6} = \frac{1.04\,(26.4+x)}{30}$ or, x = 0.6 m.

Substituting the value of x in the equation of y, the value of y = 0.936.

Area of the influence line diagram $= \frac{1}{2}(1.04 + 0.936) \times 3.6 = 3.56$

Force in the diagonal = Area of I.L. × UDL = 3.56 × 9.72 = 34.6 tonnes (Compression).

Considering the impact and distribution factor, maximum force in the diagonal AL

= 1.1 × 1.2 × 34.6 = 45.67 tonnes (Compression)

Force in bottom chord AB

Ordinate of the influence line diagram $= \frac{5a}{6h} = \frac{5\times6}{6\times8} = 0.625$

As before $y = \frac{(12-x)}{12} \times 0.625.$

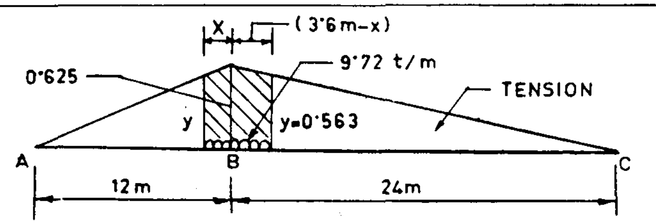

FIG. 14'17 (Ex.14·4)

Also $y = \frac{20.4 + x}{24} \times 0.625 \quad \therefore \quad \frac{12 - x}{12} = \frac{20.4 + x}{24}$ or, $x = 1.2$ m. $\therefore$ $y = 0.563$

Area of the influence line diagram under the tracked load $= \frac{1}{2}(0.625 + 0.563) \times 3.6 = 2.14$.

Force in bottom chord = Area of IL × UDL = 2.14 × 9.72 = 20.80 tonnes (tension). Maximum force in bottom chord considering impact and distribution factor = 1.1 × 1.2 × 20.80 = 27.46 tonnes (tension).

14.6 REFERENCES

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CHAPTER 15

STEEL-CONCRETE COMPOSITE BRIDGES

15.1 GENERAL

15.1.1 If a bridge deck consists of R. C. slab simply resting on few steel girders, the R. C. deck slab will take the superimposed load and live load by spanning between the steel girders and thus will transfer the loads on to the steel girders. The steel girders, on the other hand, will have flexure in the longitudinal direction and transfer the loads from the bridge deck on to the abutments or piers. In such bridge decks, the bending moment caused by the loads from the bridge deck is resisted by the steel girders themselves without having any assistance from the deck slab for the fact that separation and slip due to longitudinal shear occur at the junction of deck slab and steel girders. Therefore, the two units viz. the deck slab and the steel girder cannot act monolithically in unison as a single unit.

15.1.2 The aforesaid two units can be made to act as one unit thereby providing greater moment of intertia and thus greater sectional modulus if by some mechanical device, the separation and the slip at the interface between the deck slab and the steel girders are prevented. The mechanical device is known as *"shear connectors"* and in such bridge decks, the depth of the girders is reckoned from the bottom of girders to top of slab, the deck slab acting as a top flange of the new girders termed as *"composite girders"*. Since the deck slab takes the major part of the compressive force, the bottom flange of the steel girder has to be increased suitably to take the tensile force.

15.1.3 The advantages of composite girders are :

1. The load carrying capacity of steel girders can be increased to a great extent if some amount of tensile steel is added to the bottom flange and the girder is made monolithic with the deck slab.
2. Combination of in-situ and prefabricated units and thus saves form work and costly staging.
3. Quicker in construction as no staging is required to be made for the casting of deck slab, if desired.

15.2 SHEAR CONNECTORS

15.2.1 There are two types of shear connectors viz. rigid shear connectors consist of short length square or rectangular bars, stiffened angles, channels or tees, welded on to the top flange of the steel girders (Fig. 15.1). These shear connectors prevent the slip by bearing against the concrete of the deck slab. To prevent vertical separation between the top of girder and slab, anchorage device as shown in Fig. 15.3 shall be provided for all the shear connectors shown in Fig. 15.1.

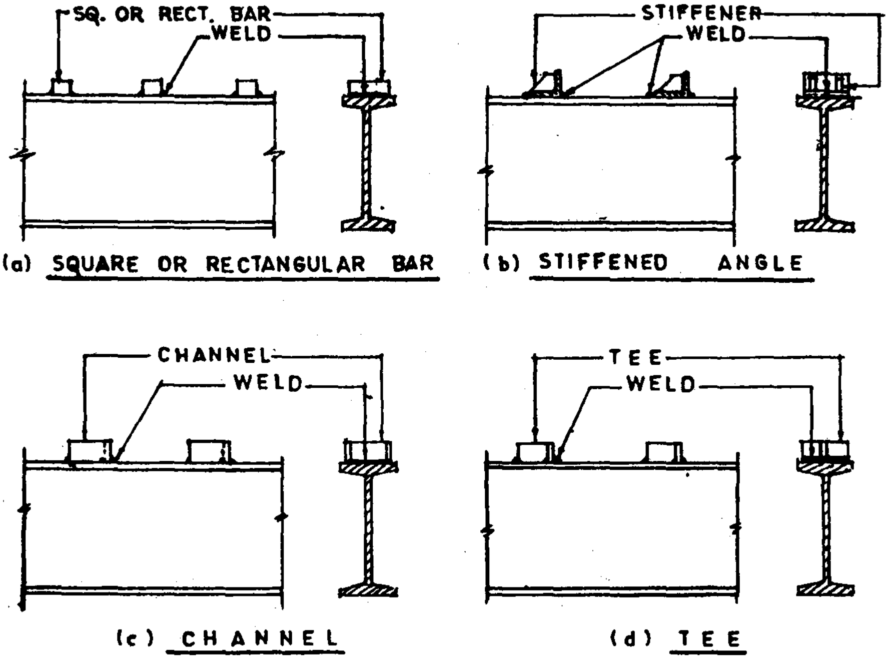


FIG. 15·1 TYPES OF RIGID SHEAR CONNECTORS (IRC)[2]

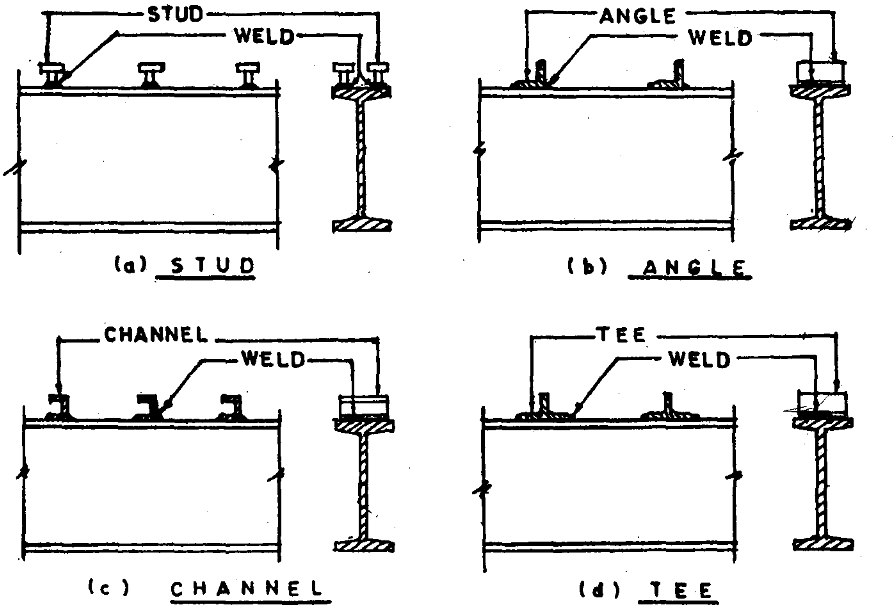
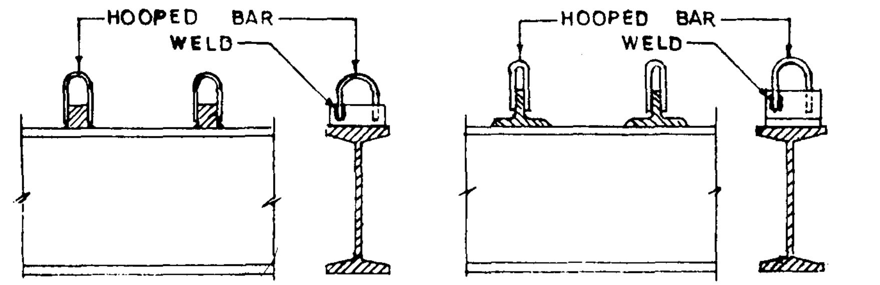


FIG. 15·2 TYPES OF FLEXIBLE SHEAR CONNECTORS (IRC)[2]



(a) BAR RIGID CONNECTOR (b) TEE FLEXIBLE CONNECTOR

FIG. 15·3 ANCHORAGE DEVICE FOR SHEAR CONNECTORS (IRC)[2]

15.2.2 Flexible shear connectors consist of studs, angles, channels and tees welded on to the top flange of the steel girders (Fig. 15.2). These shear connectors offer the resistance by bending. As in rigid shear connectors, anchorage device shall be provided in some of the flexible shear connectors where it is necessary to prevent the separation viz. in the types shown in Fig. 15.2b and 15.2d. The head of the studs (Fig. 15.2a) or the horizontal leg of the channel (Fig. 15.2c) provides the necessary anchorage and as such no separate anchorage device is necessary in these cases.

15.3 DESIGN PRINCIPLES

15.3.1 In a non-composite steel girder, the top flange takes the compressive force and the bottom flange, the tensile force caused by bending of the girder due to superimposed loads. The deck slab does not take any longitudinal stress on account of bending of the girder. In the composite girder, however, the top flange of the steel girder as well as the R. C. deck slab resist the compressive force, the bottom flange taking the tensile force as usual. As a result of having larger compression area, the steel girder possesses higher load carrying capacity when the area of the bottom flange of the steel girder is increased.

15.3.2 Equivalent Area of Deck Slab

Since the steel girder and the R. C. deck slab are made of materials having different modulus of elasticity, the area of the deck slab is required to be converted into equivalent steel area. For this purpose, the depth of the slab is kept unchanged and the effective flange width as determined from Art. 15.3.3 is reduced by dividing the effective width by the modular ratio, m, given by : $m = \frac{E_s}{E_c}$

Where E_s = Modulus of elasticity of steel of girder.

E_c = Modulus of elasticity of concrete of deck slab.

15.3.3 Effective Flange Width

The effective flange width of T or L beams shall be least of the following :

a) *In case of T-beams*

i) One fourth the effective span of the beam.

ii) The breadth of the web plus twelve times the thickness of the slab.

b) *In case of L-beams*

i) One-tenth the effective span of the beams.

ii) The breadth of the web plus one-half the clear distance between the webs.

iii) The breadth of the web plus six times the thickness of the slab.

15.3.4 Equivalent Section

The sectional properties required for the evaluation of stresses in the girder are obtained on the basis of the equivalent section of the composite girder determined in terms of Art. 15.3.2.

15.3.5 Design Assumptions

The composite girders are designed on the basis of any one of the following assumptions :

i) The steel girders are adequately proped at least at mid-span and the quarter spans before the form-work is made and the deck slab is cast. When the deck slab after casting has gained strength at least upto 75 percent of the characteristic strength, the wheel-guard, footway slab, railing, wearing etc. may be cast after removal of props. In this case, only the self weight of the steel girders is carried by the non-composite section and all other dead and live loads are carried by the composite section..

ii) After the erection of the steel girders, the form work for deck slab is supported over the steel girders (unpropped) and the deck slab is cast. After 75 percent maturity of the deck slab concrete, the item like footway slab, wheel-guard, railing and wearing course are cast. In such case, the dead load of the steel girders and the deck slab including its form-work is carried by the non-composite steel girders but the second stage of dead loads and live loads are carried by the composite section.

15.3.6 Design for Flexure

The bending moments induced by the loads on the non-composite steel girders as stated in Art. 15.3.5 shall be resisted by the non-composite section and those due to loads coming on the composite section shall be resisted by the composite section. For this purpose, the sectional properties of the composite section shall be determined as explained in Art. 15.3.4.

15.3.7 Design for Shear

15.3.7.1 The vertical shear shall be resisted by the steel girder only.

15.3.7.2 The longitudinal shear at the interface between steel girder and the deck slab shall be calculated by the following formula :

$$V_L = \frac{V \cdot A_C \cdot Y}{I} \tag{15.1}$$

Where V_L = Longitudinal shear at the interface per unit length.

V = Vertical shear due to dead load placed after composite action is effective and live load including impact.

A_C = Transformed compressive area of concrete above the interface.

Y = Distance from the neutral axis of the composite section to the centroid of the area A_C under consideration.

I = Moment of inertia of the composite section.

15.3.7.3 The longitudinal shear at the interface shall be resisted by the shear connectors and adequate transverse shear reinforcement.

15.3.8 Differential Shrinkage

The concrete deck slab after casting over the steel girders will have a tendency to shrink as in all concrete members. At the initial stage when the concrete is green, some shrinkage takes place but from the time the concrete gains strength, the shrinkage is prevented by the shear connectors provided at the interface since the top flange of the steel girder does not shrink. This causes the differential shrinkage and tensile stress is developed in the longitudinal direction in the deck slab. To cater for differential shrinkage stresses, minimum tensile reinforcement in the longitudinal direction in the deck slab shall be provided which shall not be less than 0.2 percent of the cross-sectional area of the slab.

15.3.9 Design of Shear Connectors

The design of shear connectors shall be done in accordance with the provisions of Clause 611.4 of IRC : 22-1986 (Section VI – Composite Construction).

15.3.10 Design of Transverse Reinforcement

The longitudinal shear at the interface is prevented by the shear connectors which derive strength either by bearing against concrete of deck slab (rigid shear connectors), or by bending against the concrete (flexible shear connectors) as explained in Art. 15.2. But the concrete around the shear connectors may fail by shearing by formation of shear planes as shown in Fig. 15.4a to 15.4d. The failure of this kind may be prevented by provision of transverse shear reinforcement as shown in Fig. 15.4. For further details, Clause 611.5 of IRC : 22-1986 (Section VI – Composite Construction) may be consulted.

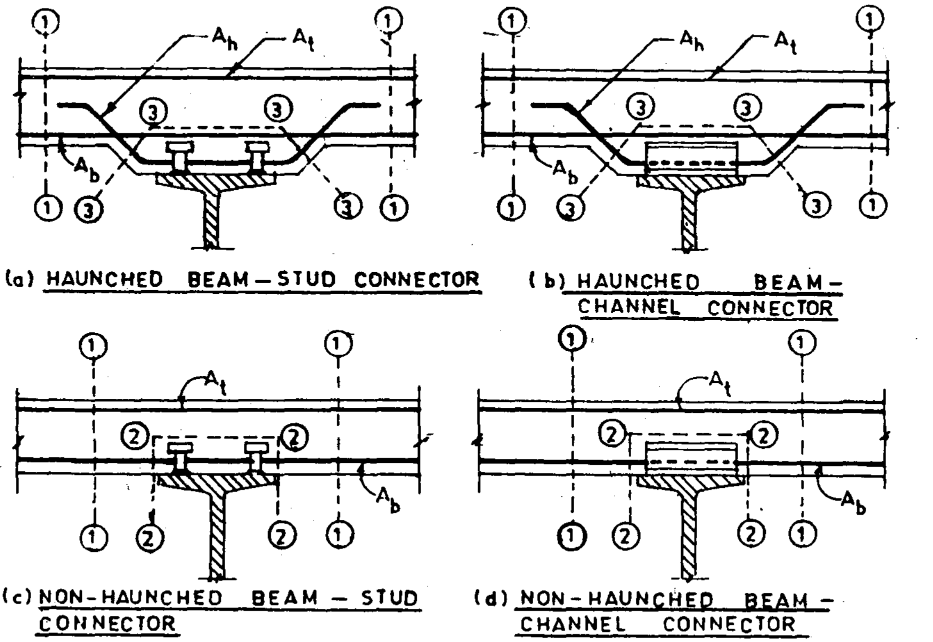


FIG. 15·4 SHEAR PLANES & TRANSVERSE SHEAR REINFORCEMENT (IRC)[2]

15.4 DETAILING

Minimum dimensions for haunches to be provided in composite deck of the type shown in Fig. 15.4b shall be provided as per Clause 612.2 of IRC : 18-1986. Dimensions and other details of the shear connections shall be as per provisions of Clause 612.2 to 612.4 of IRC : 18-1986.

ILLUSTRATIVE EXAMPLE 15.1

A highway bridge of 12m span is to be designed as a composite deck consisting of 200 mm. thick R. C. deck slab of M 20 concrete and 4 Nos. steel girders. The details of the deck are shown in Fig. 15.5. The bridge shall be designed for single lane of IRC Class 70 R or two lanes of Class A loading on the assumption of Art. 15.3.5 (ii). Design and detailing of the following items shall be done :

*i) **Flexural resistance of the composite section and steel section of the composite girder.***

*ii) **M. S. Stud shear connections which are proposed to be used in the bridge.***

*iii) **Transverse shear reinforcement.***

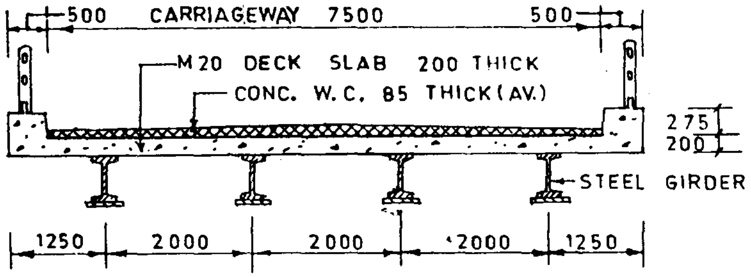


FIG. 15·5 DETAILS OF DECK (Example 15·1)

Solution

Step 1 Dead Load of deck per metre.

(a) ***First Stage (only deck slab over compound section)***

i) Wt. of deck slab = $8.5 \times 1.0 \times 0.2 \times 2400$	4,080 Kg.

(b) ***Second Stage (Balance DL)***

i) Safety Kerb etc. = $2 \times 0.5 \times 0.275 \times 1.0 \times 2400$	660 Kg.
ii) Parapet Kerb = $2 \times 0.2 \times 0.25 \times 1.0 \times 2400$	240 Kg.
iii) Railing ... 2×150 Kg.	300 Kg.
iv) 85 mm. thick (average) wearing course = $7.5 \times 0.085 \times 1.0 \times 2500$	1,595 Kg.
	2,795 Kg.

Step 2 Dead Load Moments

Total D. L. = 4080 + 2795 = 6875 Kg./m.

Assume weight of steel girder including shear connector @ 15% of total D. L. (approx.) = 985 Kg/m.

Total 1st Stage D. L. = 4080 + 985 = 5065 Kg/m.

Total 2nd Stage D. L. = 2795 Kg/m.

Assuming uniform sharing, load per girder is 1266 Kg/m and 700 Kg/m for 1st and 2nd stage dead load.

D. L. M. per girder for 1st Stage D. L. = $1266 \times \frac{(12.0)^2}{8} = 22{,}780$ Kgm..

D. L. M. per girder for 2nd Stage D. L. = $700 \times \frac{(12.0)^2}{8} = 12{,}600$ Kgm.

Step 3 Live Load Moments

Since the span of the bridge is the same as the span of the T-beam bridge in Illustrative Example 8.1 in Chapter 8, the live load moments for the latter bridge may be adopted for the composite bridge also.

Maximum L. L. moment with impact for single lane of Class 70 R loading = 1,87,000 Kgm.

Average L. L. moment per girder $= \frac{1{,}87{,}000}{4} = 46{,}750$ Kgm.

The distribution coefficient for outer girder as obtained for T-beam bridge is 1.45. Let a value of 1.50 may be taken in this case since the distance of the outer girder is more for composite deck than that for the T-beam deck.

∴ Design L. L. moment for outer girder = $1.5 \times 46{,}750 = 70{,}125$ Kgm.

Step 4 Design of Section

It is given that the form-work for deck slab will be done from the steel girders placed in position before casting of deck and no props will be placed below the steel girders. Therefore, the steel sections shall resists the moment due to its own weight as well as the weight of the deck-slab including the weight of form-work and construction live load as mentioned in Art. 15.3.5 (ii).

Therefore, the design moments for non-composite sections are :

i) Due to self wt. of steel girders and deck slab	22,780 Kgm.
ii) Add 10% for weight of form work etc.	2,280 Kgm.
	25,060 Kgm.

Design moment for composite section

The stresses induced in the compound section of the steel girder due to first stage D. L. design moments are to be added to the stress in the composite section induced by the second stage dead load and L. L. moment.

∴ Design moment = Second Stage D. L. moment + L. L. moment = 12,600 + 70,125 = 82,725 Kgm.

As stated in Art. 15.1.2, the composite steel girder will have more area for bottom flange than that of the top flange and as such the steel section will be unsymmetrical about horizontal axis. This will be achieved by providing additional plate to the bottom flange of a symmetrical R. S. J. the section of which may be determined approximately on the basis of one-third of the total D. L. and L. L. moment i.e.,

$$\frac{1}{3} \times (25{,}060 + 82{,}725) = 35{,}930 \text{ Kgm..}$$

Assuming a steel stress for M. S. steel girder as 1500 Kg/cm^2,

Section modulus of the symmetrical R. S. J. $= \frac{35{,}930 \times 10^2}{1500} = 2395 \text{ cm}^3$

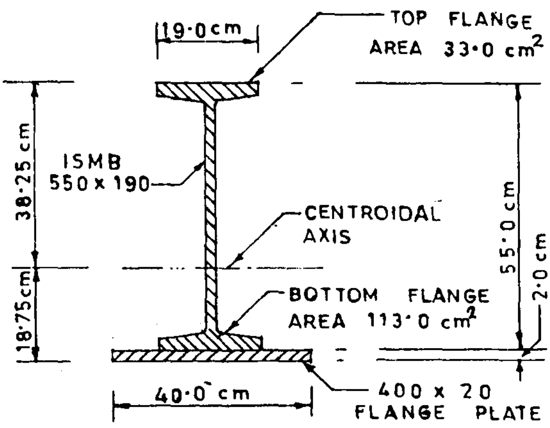


FIG. 15·6 COMPOUND STEEL SECTION (Ex. 15·1)

ISMB 550 × 190 has a section modulus of 2360 cm^3. (Area = 132 cm^2 and weight per metre = 104 Kg) (Fig. 15.6).

Mr. J. C. Hacker has suggested the following empirical formulae for the determination of the trial steel section.

$$R = \frac{Ast}{Asb} = \frac{15.0}{60 - L}, \quad \text{and} \tag{15.1}$$

$$Asb = \frac{1}{6_s}\left[\frac{M_{DL}}{d} + \frac{(M'_{DL} + M_{LL})}{d + t}\right] \tag{15.2}$$

Where Ast = Top flange area, cm^2
Asb = Bottom flange area, cm^2
L = Span, m
6_s = Permissible steel stress, Kg/cm^2
M_{DL} = Moment due to 1st stage dead load, Kgcm.
M'_{DL} = Moment due to 2nd stage dead load, Kgcm.
M_{LL} = Moment due to live load, Kgcm.
d = Depth of symmetrical girder, cm.
t = Depth of deck slab, cm.

From equation 15.1, $R = \frac{Ast}{Asb} = \frac{15.0}{60 - 12} = 0.3125$

From equation 15.2,

$$Asb = \frac{1}{1500}\left[\frac{25{,}060 \times 100}{55.0} + \frac{(12{,}600 + 70{,}125) \times 100}{(55.0 + 20.0)}\right]$$

$$= \frac{1}{1500}(45{,}560 + 1{,}10{,}300) = 103.90 \text{ cm}^2$$

$$Ast = R.\ Asb = 0.3125 \times 103.90 = 32.47 \text{ cm}^2$$

Ast. available in the R. S. J. = 33.0 cm^2 (Fig. 15.5). Using 40 cm × 2 cm plate at the bottom flange, Asb = (40 × 2 + 33) = 113.0 cm^2, total area of the compound steel girder = (132 + 40 × 2) = 212 cm^2 and total weight = 167 kg/m.

Step 5 Centroidal Axis of the Compound Steel Section

Referring to Fig. 15.5 and taking moment from bottom, $\bar{x} \times 212 = (40 \times 2.0 \times 1.0 + 132.0 \times 29.5) = 3974$

$$\therefore \; \bar{x} = \frac{3974}{212} = 18.75 \text{ cm. from bottom.}$$

Step 6 Moment of Inertia of the Compound Section

i)	R. S. J. (own axis)	64,890 cm^4
ii)	Bottom plate (own axis) = $\frac{40 \times (2.0)^3}{12}$	30 cm4
iii)	R. S. J. (centroidal axis) = $132 \times (29.5 - 18.75)^2$	15,250 cm^4
iv)	Bottom Plate (centroidal axis) = $40 \times 2.0 \times (18.75 - 1.0)^2$	25,200 cm^4
	Total I =	1,05,370 cm^4

$$\therefore \; Z_{tg} = \frac{1,05,370}{38.25} = 2755 \text{ cm}^3; \quad Z_{bg} = \frac{1,05,370}{18.75} = 5620 \text{ cm}^3$$

Step 7 Stresses in the Compound Steel Section due to self wt. of girder plus weight of slab, form work etc.

$$M_{DL} = 25,060 \times 100 \text{ Kgcm.}$$

$$\therefore \; 6_{tg} = \frac{25,060 \times 100}{2755} = (+)\,909.62 \text{ Kg/cm}^2; \qquad 6_{bg} = \frac{25,060 \times 100}{5620} = (-)445.91 \text{ Kg/cm}^2$$

Permissible steel stress = 1500 kg/cm^2. Hence the steel stresses remain within permissible limit when the compound section acts as non-composite section.

Step 8 Equivalent Area of the Composite Section

The composite section consisting of R. C. deck slab and steel girder as shown in Fig. 15.7 is to be converted into equivalent steel section as explained in Art. 15.3.2. This again is dependent on the effective flange width of the composite section to be determined in accordance with Art. 15.3.3.

Effective flange width is the least of the following :

i) $\frac{1}{4} \times \text{span} = \frac{1}{4} \times 12.0 = 3.0$ m. = 300 cm.

ii) the distance between centre of web of the beam = 200 cm.

iii) breadth + 12 × thickness of slab = 1.0 + 12 × 20 = 241 cm.

Hence 200 cm. is the least value and as such the effective flange width.

Equivalent width from Art. 15.3.2 = $\frac{\text{Effective flange width}}{m} = \frac{200}{10} = 20.0$ cm.

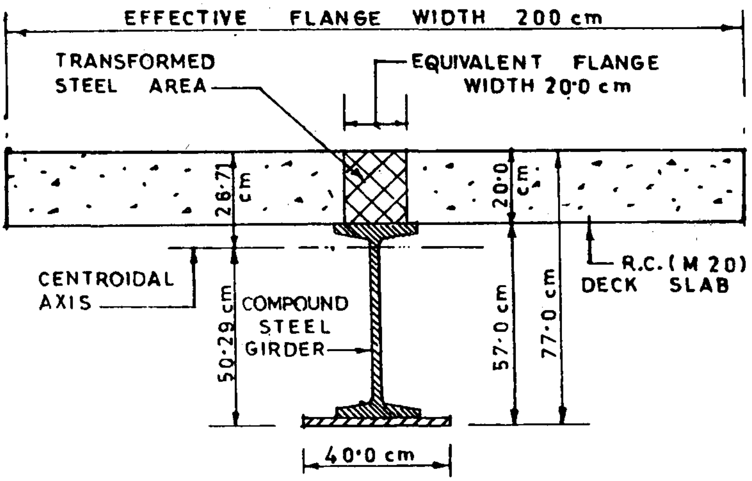


FIG. 15·7 COMPOSITE SECTION((STEEL & CONCRETE)(Ex.15·1)

Hence area of the composite section = Area of compound steel section + equivalation steel area of deck slab. $= 212 + 20 \times 20.0 = 612\ \text{cm}^2$

Step 9 Centroidal Axis of the Equivalent Composite Section

Taking moment about the bottom of girder, $\bar{x}_1 \times 612 =$ Area of compound steel section × its CG distance from bottom + Area of concrete section (transformed steel area) × its C. G. distance from bottom. $= 212 \times 18.75 + 20 \times 20 \times 67.0 = 30{,}775\ \text{cm}^3$.

$$\therefore\ \bar{x}_1 = \frac{30{,}775}{612} = 50.29\ \text{cm}$$

Step 10 Moment of Inertia of the Equivalent Section

i)	Compound Steel Section (own axis) (Step 6)	1,05,370 cm^4
ii)	Concrete section (transformed steel area) (own axis) $= \frac{20.0 \times (20)^3}{12}$	13,300 cm4
iii)	Compound Steel Section (Centroidal axis) $= 212\ (50.29 - 18.75)^2$	2,10,900 cm^4
iv)	Concrete Section (transformed steel area) (Centroidal axis) $= 20.0 \times 20 \times (26.71 - 10.0)^2$	1,11,700 cm^4
	Total Ic	4,41,300 cm^4

$$\therefore \text{Section modulus of top of slab, } Z_{ts} = \frac{4{,}41{,}300}{26.71} = 16{,}520\ \text{cm}^3$$

$$\text{Section modulus of top of girder, } Z_{tg} = \frac{4{,}41{,}300}{6.71} = 65{,}770\ \text{cm}^3$$

$$\text{Section modulus of bottom of girder, } Z_{bg} = \frac{4{,}41{,}300}{50.29} = 8775\ \text{cm}^3$$

Step 11 Stresses due to 2nd stage Dead Load and Live Load Moment on the Composite Section

Stress at top of slab, $6_{ts} = \dfrac{M'_{DL} + M_{LL}}{Z_{ts} \times m} = \dfrac{82{,}725 \times 100}{16{,}520 \times 10} = (+)\ 50.08\ \text{Kg/cm}^2$

Stress at top of steel girder $= \dfrac{M'_{DL} + M_{LL}}{Z_{tg}} = \dfrac{82{,}725 \times 100}{67{,}770} = (+)\ 122.07\ \text{Kg/cm}^2$

Stress at bottom of steel girder $= \dfrac{M'_{DL} + M_{LL}}{Z_{bg}} = \dfrac{82{,}725 \times 100}{8775} = (-)\ 942.74\ \text{Kg/cm}^2$

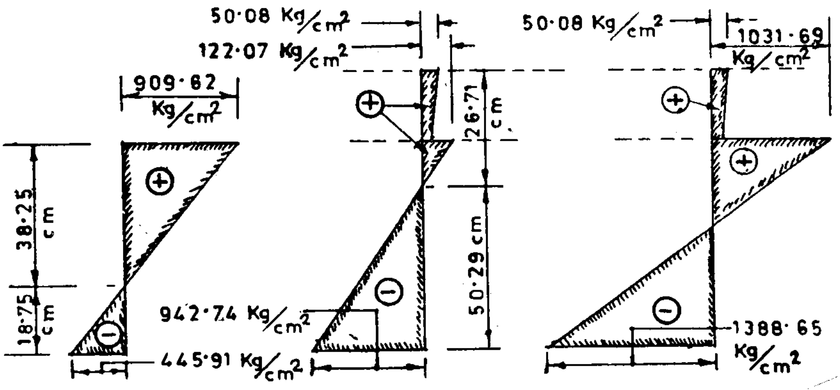


FIG. 15·8 STRESSES IN COMPOSITE SECTION (Example 15·1)

Step 12 Final Stresses in the Composite Girder

The final stresses in the girder and deck slab due to longitudinal bending to sustain all the dead and live loads are as shown in Table 15.1 and in Fig. 15.8 for better understanding.

TABLE 15.1 : FINAL STRESSES IN THE COMPOSITE GIRDER

Stress at	Stresses (Kg/cm²) due to		
	First Stage D.L.	Second Stage D.L. + L.L.	Total Load
Top of slab	—	+ 50.08	+ 50.08
Top of girder	+909.62	+122.07	+ 1031.69
Bottom of girder	– 445.91	– 942.74	– 1388.65

Note : Permissible concrete stress in bending for M20 concrete is 6.7 MPa i.e. 68.36 Kg/cm² and permissible steel stress in bending for structural steel conforming to IS:226 is 1500 Kg/cm² (+) indicates compression and (–) indicates tension.

Step 13 Design of Shear Connectors

The shear connectors will start functioning when the concrete of the deck slab gains maturity. Therefore, the shear at the ends of the girders due to self-weight of compound steel girders and 1st stage of dead loads i.e. the weight of the green concrete of the deck slab including its form work will have no effect on the shear connectors. Only the shear due to 2nd stage of dead load and live load will cause longitudinal shear at the interface and as such will need shear connectors to resist the slip. D.L. Shear due to 2nd Stage of dead load $= \frac{1}{2} \times 2795 \times 12.0 = 16{,}770$ Kg.

Assuming equal sharing, shear per girder $= \frac{16{,}770}{4} = 4{,}190$ Kg.

Live load shear (single lane of Class 70R loading — Refer Fig. 8.10) = 56,670 Kg.

For 12 m span, impact factors for steel and concrete bridges are 25 per cent and 10 per cent respectively (Refer Art. 5.6.3 of Chapter 5). The instant bridge is a combination of steel and concrete and as such an average impact factor may be considered in the design of shear connectors.

$\therefore$ Average impact factor $= \frac{1}{2}(10 + 25) = 17.5\%$

$\therefore$ L.L. shear with impact $= 1.175 \times 56{,}670$ Kg. $= 66{,}590$ Kg

Shear for intermediate girders will be maximum and this is evaluated as per Clause 305:9.2 (i) of IRC:21-1987. The sharing of shear may be taken as 0.35 for each intermediate girder $= 0.35 \times 66{,}590$ Kg $= 23{,}300$ Kg.

Fig. 15.9 shows the S.F. diagram for one intermediate girder. From Fig. 15.9c, the total vertical shear due to dead load placed after composite action is effective and live load with impact near the support is 27,490 Kg.

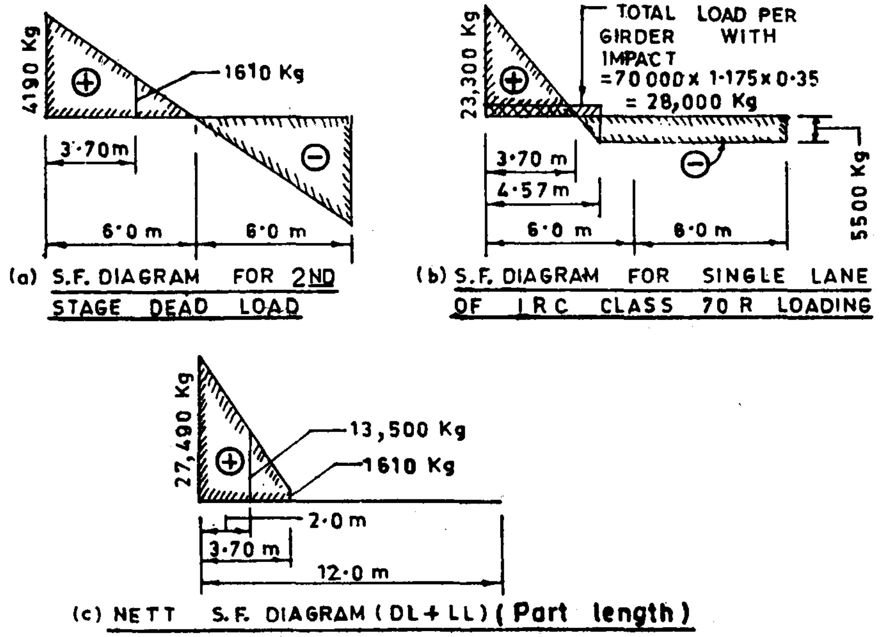


FIG. 15·9 SHEAR FORCE DIAGRAM (Example 15·1)

Shear connector near support —

As per Art. 15.3.7.2, the longitudinal shear, V_L per unit length at the interface is given by,

$$V_L = \frac{V \cdot Ac \cdot Y}{I} = \frac{27{,}490 \times (20 \times 20) \times 6.71}{4{,}41{,}300} = 167.19 \text{ Kg/cm.}$$

As per Clause 611.4.1.3.1 of IRC:22-1966 (Section VI – Composite Construction), the safe shear value of each mild steel (minimum UTS of 460 MPa, and yield point of 350 MPa and elongation of 20 percent) is given by,

$$Q = 4.8\, h \cdot d \sqrt{Fck} \text{ for a ratio of } \frac{h}{d} < 4.2$$

Where Q = Safe shear resistance in Kg. of one shear connector.
h = Height of stud in cm.
d = Dia. of stud in cm.
Fck = Characteristic strength of concrete in Kg/cm^2.

Using 20 mm. dia 100 mm. high stud, $Q = 4.8 \times 10 \times 2 \sqrt{200} = 1350$ Kg.

If two shear connectors are placed in one transverse line, shear resistance of 2 shear connectors $= 2 \times 1350 = 2700$ Kg.

Hence spacing $= \frac{2700}{167.19} = 16.14$ cm. Say150mm.

Design shear at 2.0 m. from support (Fig. 15.9c) = 13,500 Kg., i.e., nearly half of shear at support.

Hence, the spacing of shear connectors is twice the previous value, i.e., 300 mm. A spacing of 200 mm. may be used in this case.

Shear at centre = 5500 Kg (Fig. 15.9b).

Hence, spacing of shear connectors (inversely proportional to vertical shear and spacing near support) $= 160 \times \frac{27{,}490}{5{,}500} = 800$ mm.

Use a spacing of 300 mm. from practical consideration. The spacing of shear connectors throughout the length of the beam is shown in Fig. 15.10 considering that max. shear near support comes down quickly.

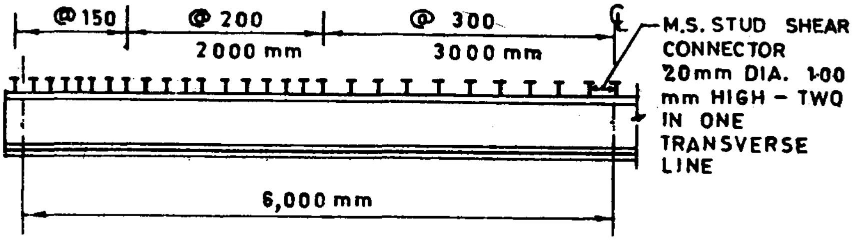


FIG. 15·10 ARRANGEMENT OF SHEAR CONNECTOR (Ex 15·1)

Step 14 Design of Transverse Shear Reinforcement

The longitudinal shear force, V_L per unit length transferred from the steel girder to the deck slab through any shear plane shall not exceed either of the following and transverse shear reinforcement shall be provided accordingly (Refer clause 611.5 of IRC:22-1986 – Section VI).

$$\text{(i)}\quad 0.4\, L_s \sqrt{fck} \tag{15.3}$$

$$\text{or}\quad \text{(ii)}\quad 0.7\, A_s\, 6_y + 0.08 L_s \sqrt{fck} \tag{15.4}$$

Where L_s = The length of shear plane under consideration in mm as shown in Fig. 15.4.

f_{ck} = Characteristic strength of concrete in MPa but not more than 45 MPa

A_s = The sum of the cross-sectional areas of all reinforcing bars intersected by the shear plane per unit length of the beam (mm^2/mm). This includes those provided for flexure.

6_y = The yield stress (MPa) of the reinforcing bars intersected by the shear plane but not more than 450 MPa.

In the instant case, the shear planes will be 1-1 and 2-2 as shown in Fig. 15.4a. L_s in case of shear plane 1-1 = $2 \times 200 = 400$ mm. and L_s in case of shear plane 2-2 = $(190 + 2 \times 100) = 390$ mm. A value of 400 mm may be taken in the design. V_L near support has already been evaluated while designing the shear connector which is equal to 167.19 Kg/cm = 164 N/mm.

Minimum transverse reinforcement as per Clause 611.5.2.3 of IRC:22-1986 (Section VI) is given by,

$$A_s = \frac{0.8\, L_s}{6_y} \tag{15.5}$$

$$\text{Equation 15.5 gives, } A_s = \frac{0.8 \times 400}{415} = 0.77 \text{ mm}^2/\text{mm}.$$

Top and bottom bars provided for bending in case of slab and girder bridge (Fig. 8.5) are 12 Φ @ 220 mm. In the present case the bars will be similar in quantity. As per Clause 6.11.5.2.3 :

For shear plane 1-1, As = (At + Ab) but At or Ab shall be 50% of As. For shear plane 2-2, As = 2 Ab.

In the instant case, As available both for shear plane 1-1 and 2-2 is $\frac{2 \times 113}{220} = 1.03$ mm^2/mm. against the minimum requirement of 0.77 mm^2/mm.

As per equation 15.3, the shearing resistances $= 0.4\, L_s \sqrt{fck} = 0.4 \times 400 \times \sqrt{20} = 716$ N/mm.

As per equation 15.4, the shearing resistance $= 0.7\, As\, 6_y + 0.08\, L_s \sqrt{fck}$

$$= 0.7 \times \frac{2 \times 113}{220} \times 415 + 0.08 \times 400 \sqrt{20}$$

$$= 298 + 143 = 441 \text{ N/mm.}$$

The longitudinal shear V_L at the interface per mm. is 164 N/mm. which is much less that the shearing resistance of the shear planes. Hence safe.

The detailing of the transverse shear reinforcement is shown in Fig. 15.11.

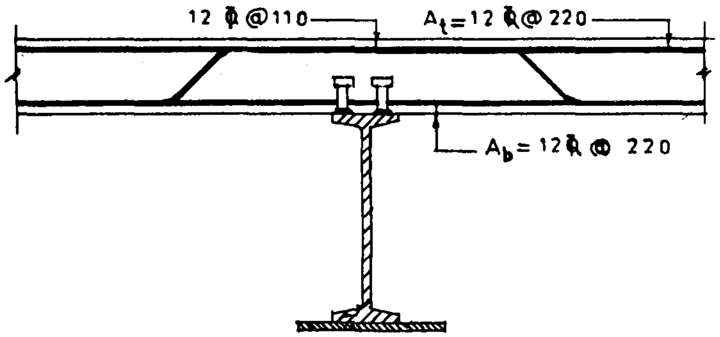


FIG. 15·11 DETAILS OF TRANSVERSE SHEAR REINF. IN STEEL-CONC. COMPOSITE BEAM (Ex 15·1)

15.5 REFERENCES

1. IRC : 24-1967 Standard Specifications and Code of Practice for Road Bridges, Section V — Steel Road Bridges, Indian Roads Congress.
2. IRC : 22-1966 Standard Specifications and Code of Practice for Road Bridges, Section VI Composite Construction, Indian Roads Congress.
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4. Hacker, J. C. — *"A Simplified Design of Composite Bridge Structures"* — Paper No. 1432 of the Proceedings of ASCE, November, 1957.
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Chapter **16**

PRESTRESSED CONCRETE BRIDGES

16.1 GENERAL

16.1.1 Prestressed concrete is that concrete in which internal stresses are so induced by the application of some special technique that the stresses so developed are of opposite nature to those produced by the external loads such as dead and live loads which the member is to carry and for which the member is to be designed. By prestressing, the strength of a member can be greatly increased since a part of the stresses developed by the dead and live loads is nullified by the prestressing force.

16.1.2 The fundamental difference between reinforced and prestressed concrete is illustrated in Fig. 16.1. In reinforced concrete members, there will be no stress in it when it is unloaded but when loaded, compressive and tensile stress will develop at top and bottom fibre respectively. On the other hand, in a prestressed concrete member when the prestress is applied but it is unloaded, compressive stress will be induced at bottom fibre, the stress at the top fibre being zero or marginally tensile or compressive, depending on the eccentricity of the

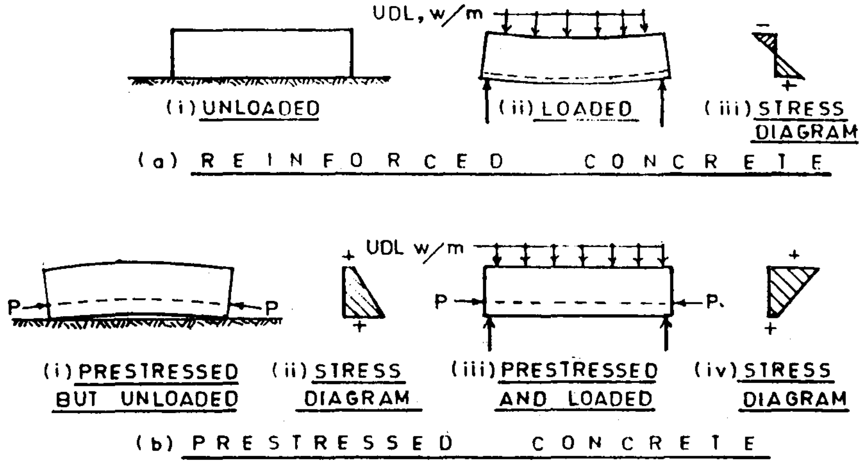


FIG.16·1 STRESSES IN REINFORCED AND PRESTRESSED CONCRETE BEAMS prestressing force. The stresses induced by the loads will be partly balanced by the stresses of opposite nature already produced by the prestressing force.

16.2 ADVANTAGES OF PRESTRESSED CONCRETE

16.2.1 The development of prestressed concrete has opened new vistas in the construction of highway bridges. Prestressed concrete bridges have many advantages over the reinforced concrete ones and therefore, majority of long span concrete highway bridges are now-a-days being constructed of prestressed concrete. These bridges need less quantity of steel, concrete and formwork. Less concrete in girders reduces the dead load moments and shears. Moreover, prestressed girders being lighter, launching of girders becomes possible in flowing streams where staging is not possible or cost of staging will be tremendously high. In addition, due to the reduced weight of the prestressed girders and slab, it is possible to reduce the cost of substructure and foundation causing thereby overall economy of the bridge. Prestressed concrete sections have further advantage that the full section remains in compression eliminating thereby any possibility of tension cracks and that the inclined prestressed tendons reduce the shear force at the ends thus resulting in saving of shear reinforcement.

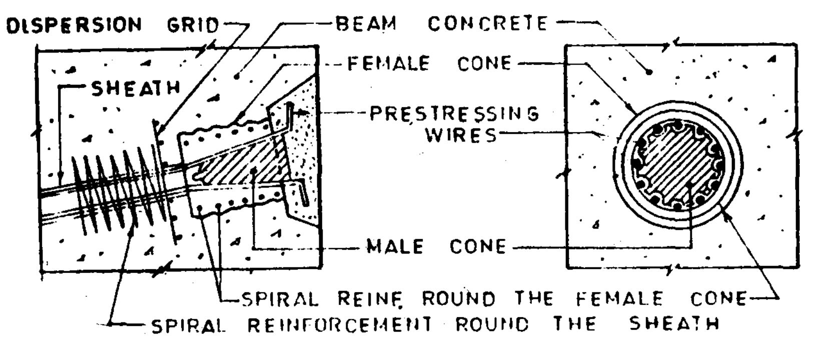
16.3 SYSTEMS OF PRESTRESSING

16.3.1 In prestressed bridge construction, post-tensioning method is generally adopted and as such only post-tensioning will be dealt with in this chapter.

16.3.2 The following prestressing systems are very commonly employed in India for this sort of construction. It may be mentioned in this connection that the main difference in different systems of prestressing lies in the principle by which the prestressing bars or cables are stressed and anchored to the concrete members otherwise there is not much difference either in the design procedure or in the construction method.

16.3.3 Freyssinet System

This system anchors the prestressing cables by wedge action with the help of two cones, the female cone and the male cone (Fig. 16.2). The prestressing cables generally consist of 8, 12 or 18 nos. of either 5 mm or 7 mm wires and these wires are inserted between the walls of male and female cone, stressed and then released. The recoiling tendency of the wires forces down the male cone and locks the wires by wedge action. No further recoiling of the wires is possible and these are permanently anchored to the concrete members. In addition, cement grout is injected into the space between the cable and the sheath for further safety against slippage of the cables. The cement grount also protects the cables against corrosion.



(a) SECTION THRO' CONES (b) CROSS SECTION

FIG.16·2 ANCHORAGE FOR FREYSSINET SYSTEM

Both the male and the female cones are made of high grade concrete with closely spaced spiral reinforcement. The male cone is slightly tapered in the form of wedge. The tensioning or stressing of the cables is made with the help of Freyssinet jacks specially made for the purpose. During concreting, the cables are protected with the help of metal sheath so that no bond is developed between the concrete and the prestressing steel otherwise tensioning of the prestressing steel will not be possible. Special care should be taken to make the sheath leakproof.

16.3.4 Magnel-Blaton System

This system also makes use of 5 mm. or 7 mm. wires as prestressing steel and the principle of anchoring the wires is the same as that of Freyssinet System viz. by wedge action but the main difference is that these wedges are made of steel instead of concrete and flat in shape instead of conical male cone of Freyssinet system (Fig. 16.3). These flat wedges anchor the wires by friction against the steel sandwich plates which against rest on steel distribution plates. The prestressing force from the cable is ultimately transferred to the concrete member through these distribution plates.

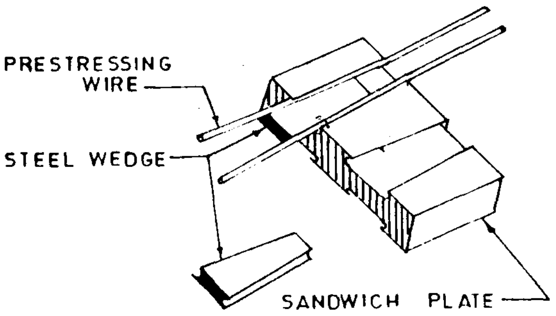
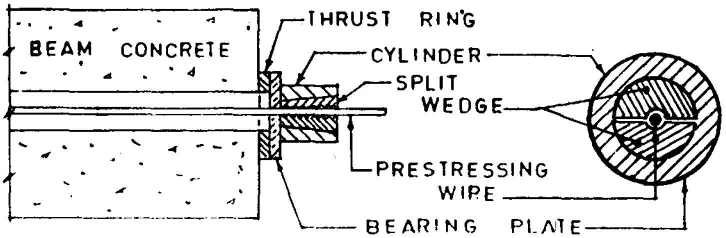


FIG.16·3 ANCHORAGE FOR MAGNEL-BLATON SYSTEM

Each steel sandwich plate can anchor 8 nos. wires. The capacity of each distribution plate is usually multiple of 8 wires. These plates may be cast in proper place on to the end block during concreting or may be laid with grout during the time of stressing. In Freyssinet system, all the wires in a cable are stressed at a time but in Magnel-Blaton system, only two wires are stressed at a time.

16.3.5 Gifford-Udall System

The diameters of wires usually used in this system are 4 mm, 5 mm and 7 mm. The anchorage unit consists of one thrust ring, one bearing plate and anchorage grips (Fig. 16.4).



(a) SECTION THRO' WEDGE (b) CROSS SECTION

FIG. 16·4 ANCHORAGE FOR GIFFORD-UDALL SYSTEM

The anchorage grip is a steel cylinder having a tapered hole inside through which split, tapered steel wedge is inserted. The wire to be anchored is passed through the steel wedge pressed between the two halves. In this sytem, each wire is anchored with independent grip and therefore, any number of wires may be arranged in each unit. The cylindrical grip bears against steel bearing plate through which a number of holes are drilled to facilitate passage of wires to be anchored. The bearing plate again bears against a thrust ring which ultimately transmits the prestressing force to the concrete member.

16.3.6 Lee-McCall System

Unlike the system mentioned in the previous paras, this system makes use of high tensile bars usually 12 mm. to 28 mm. diameter instead of wires or cables. This method is very simple in respect of anchorage unit which consists of one end plate or bearing plate and a nut (Fig. 16.5). The ends of the bars are threaded and during stressing the nuts are tightened to prevent recoiling of the stressed rod.

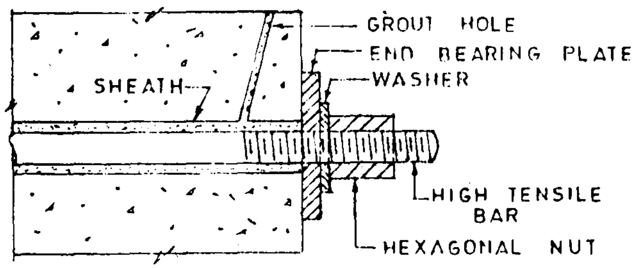


FIG.16·5 ANCHORAGE FOR LEE-Mc CALL SYSTEM

This system has the advantage over others that stressing can be done by stages as it is possible to tighten the nut at any stage. The losses of prestress due to creep, relaxation of steel etc. (most part of which occur at the early days after prestressing) can be reduced if the bars are restressed afterwards.

16.4 MODULII OF ELASTICITY

16.4.1 Modulus of Elasticity of Steel (Es)

Unless the manufacturers' certified values or test results are available, the values of modulus of elasticity of steel tendous as given in Table 16.1 may be assumed in design.

TABLE 16.1 : MODULUS OF ELASTICITY OF STEEL TENDONS (IRC)[1]

Sl. No.	Type of Steel	Modulus of Elasticity (MPa)
1	Plain hard-drawn wires (conforming to IS : 1785 and IS : 6003)	2.1×10^5
2	High tensile steel bars rolled or heat-treated (conforming to IS : 2090)	2.0×10^5
3	Strands (conforming to IS : 6006)	1.95×10^5

16.4.2 Modulus of Elasticity of Concrete (Ec)

Unless otherwise determined by tests, the modulus of elasticity of concrete shall have the following values :

$$E_c = 5700 \sqrt{fck} \text{ MPa} \quad (16.1)$$

The modulus of elasticity of concrete at j days shall be taken as :

$$E_{cj} = 5700 \sqrt{fcj} \text{ MPa} \quad (16.2)$$

16.4.3 Equivalent Flange Width

$$\text{Equivalent flange width} = \text{Effective flange width} \times \frac{Ec(s)}{Ec(p)}$$

Where Ec(s) = Modulus of Elasticity of slab.

Ec(p) = Modulus of Elasticity of precast girder.

16.5 LOSS OF PRESTRESS

16.5.1 The loss of prestress in the members occurs on account of many factors some of which are to be accounted for in designing the members and some at the time of stressing. These may be briefly stated as under:

16.5.2 Loss due to Creep in Concrete

When concrete section remains under stress, permanent strain or creep occurs in concrete which reduces the stress in the prestressing tendons. The amount of creep depends on the magnitude of stress in the section and the age of concrete at the time of application of the prestress.

The creep strain of concrete shall be taken as shown in Table 16.2.

TABLE 16.2 : CREEP STRAIN OF CONCRETE (IRC)[1]

Sl. No.	Maturity of concrete at the time of stressing as a percentage of fck.	Creep strain per 10 MPa stress
1	40	$9.4 \ 10^{-4}$
2	50	$8.3 \ 10^{-4}$
3	60	$7.2 \ 10^{-4}$
4	70	$6.1 \ 10^{-4}$
5	75	$5.6 \ 10^{-4}$
6	80	$5.1 \ 10^{-4}$
7	90	$4.4 \ 10^{-4}$
8	100	$4.0 \ 10^{-4}$
9	110	$3.6 \ 10^{-4}$

Note : (a) Creep strain for intermediate values may be interpolated linearly.

(b) The stress in concrete at the centroid of the prestressing steel shall be considered for calculation of loss of prestress.

(c) The creep strain during any interval shall be based on the average stress during the interval.

16.5.3 Lose due to Shrinkage of Concrete

Similar to creep strain, shrinkage strain diminishes the prestressing force in the prestressing tendons. The loss of prestress due to shrinkage in the concrete shall be calculated from the values of strain due to residual shrinkage as indicated in Table 16.3.

TABLE 16.3 : STRAIN DUE TO RESIDUAL SHRINKAGE (IRC)[1]

Sl. No.	Age of concrete at the time of stressing, in days	Strain due to residual shrinkage
1	3.	$4.3\ 10^{-4}$
2	7	$3.5\ 10^{-4}$
3	10	$3.0\ 10^{-4}$
4	14	$2.5\ 10^{-4}$
5	21	$2.0\ 10^{-4}$
6	28	$1.9\ 10^{-4}$
7	90	$1.5\ 10^{-4}$

Note : (a) Values for intermediate figures may be linearly interpolated.

16.5.4 Loss due to Relaxation of Steel

When high tensile steel is kept under stress, permanent strain or relaxation in steel, as is normally called, takes place due to which the prestress force in the tendon diminishes and loss in prestresses occurs. The relaxation loss depends on the stress in steel as given in Table 16.4. When manufacturers' certified values are not available, these values may be assumed in the design.

TABLE 16.4 : RELAXATION LOSS OF PRESTRESSING STEEL (IRC)[1]

Sl. No.	Initial prestress (UTS of Steel)	Relaxation loss (MPa)
1	0.5	0
2	0.6	35
3	0.7	70
4	0.8	90
Note : For intermediate values, linear interpolation may be done.		

16.5.5 Loss due to Seating or Slip of Anchorages

After the transfer of prestress to the anchorages, slip of wires or draw-in of male cone or strain in the anchorages occurs before the wires are firmly gripped. These effects, therefore, result in loss of prestress the value of which shall be as per test results or manufacturers' recommendations. As a rough guide, the slip or draw-in may be taken as 3 to 5 mm.

16.5.6 Loss due to Elastic Shortening

All the cables or wires of a prestressed member are not stressed at a time but stressing is done one after another depending on the necessity to satisfy different loading conditions. The elastic strain produced by the prestressing force applied on the concrete member causes some relaxation in the prestressing tendons which have been stressed earlier. It is evident, therefore, that due to this phenomenon, the tendon which has been stressed at the first instance will suffer maximum loss and the last one will suffer no loss. The loss due to elastic shortening shall be computed on the basis of the sequence of tensioning. However, for the purpose of design, the resultant loss of prestress of all the wires due to elastic shortening may be taken as equal to the product of the modular ratio and half the stress in concrete adjacent to the tendons averaged along the length. Alternatively, the loss of prestress may be calculated exactly based on sequence of stressing.

16.5.7 Loss due to Friction

Friction loss in prestressing force occurs in the prestressed member and varies from section to section. This loss depends on the co-efficient of friction between the prestressing tendon and the duct. The friction loss is divided into two parts :

i) ***Length effect*** — friction between the tendon and the duct (both straight).

ii) ***Curvature effect*** — due to the curvature of the tendon and the duct, friction is developed when the tendon is stressed and loss of prestress occurs.

The magnitude of the prestressing force P_x at any distance x from the jacking end after accounting for the friction losses due to both length and curvature effects may be given by the following equation —

$$P_x = P_o \cdot e^{-(KX + \mu\theta)} \qquad (16.3)$$

Where P_o = Prestress force at the jacking end.

P_x = Prestress force at some intermediate point at a distance x.

K = Length or wobble co-efficient per metre length of steel.

μ = Curvature co-efficient.

θ = Total angular change in radians from the jack end to the point under consideration.

x = Length of the straight portion of the tendon from the jacking end in metres.

e = Base of Naperian Logarithm (= 2.718).

The values of K and μ vary for different nature of steel and ducts or sheathing materials as indicated in Table 16.5 and these values may be used for the computation of friction losses.

TABLE 16.5 : VALUES OF K (WOBBLE CO-EFFICIENT) AND μ (CURVATURE CO-EFFICIENT) *(IRC)*[1]

Type of high tensile steel	Type of duct or sheath	Values recommended for	
		K per metre	μ
Wire Cables	Bright metal	0.0091	0.25
	Galvanised	0.0046	0.20
	Lead coated	0.0046	0.18
	Unlined duct in concrete	0.0046	0.45

TABLE 16.5 : VALUES OF K (WOBBLE CO-EFFICIENT) AND μ (CURVATURE CO-EFFICIENT) (IRC)[1] *Contd.*

Type of high tensile steel	Type of duct or sheath	Values recommended for	
		K per metre	μ
Uncoated stress relieved strands	Bright metal	0.0046	0.25
	Galvanised	0.0030	0.20
	Lead coated	0.0030	0.18
	Unlined duct in concrete	0.0046	0.50

Note : The values of K and μ assumed in the design shall be indicated on the drawings for guidance in selecting the materials and methods that will produce results approaching the design value.

16.5.8 The various kinds of losses to be accounted for in the design of the sections and during stressing operation are discussed. It has been observed that the losses due to creep and shrinkage of concrete and relaxation of steel generally lie between 15 to 20 percent for post-tensioned structures. The loss that occur due to slip in anchorage unit is the percentage of slip with respect to the total extension of the tendon achieved by stressing it. The magnitude of the slip in the anchorage unit depends on the type of wedge and the stress in the wire and it, therefore, transpires that the loss of prestress on this account is more for short members than for long members since the amount of slip in both cases will be the same if stress in the tendon and wedge condition remain the same in both the members.

For important bridges, the stresses in the girders shall be checked for 20 percent higher time dependent losses viz. creep, shrinkage, relaxation etc. to ensure a minimum residual compression as recommended in clause 3.2.4 of IRC : SP-33.

The friction loss for long members specially for continuous one in which the curvature of the tendons changes directions is more. An average value of 12 to 15 percent may be taken as a very rough guide.

16.6 PRELIMINARY DIMENSIONS OF T-BEAMS AND BOX-GIRDERS

16.6.1 The preliminary dimensions of the girder section should be such that they satisfy all the loading conditions both at the time of construction as well as during service. The dimensions of different parts of a girder section is illustrated in Fig. 16.6 which gives a rough guide of the girder sections. The stresses in the girder for various loading conditions may be investigated with the properties of the assumed girder section. If required, the assumed dimensions of the girder may be modified suitably to arrive at the required section. The dimensions of top flange, bottom flange and web shall be such that the prestressing cables can be accommodated with appropriate cover and spacings as per code provisions. The dimensions shown in Fig. 16.6 are based on IRC : 18-1985. However, for important bridges, the dimensions of web for T-beam and box-girders shall be as indicated in para 16.6.2 below as per IRC : SP-33.

16.6.2 The thickness of web of T-beam and box-girders shall not be less than 200 mm. plus duct diameter. For cast-in-situ cantilever construction, if the prestressing cables are anchored in the web, the thickness of web shall not be less than 350 mm. uniformly.

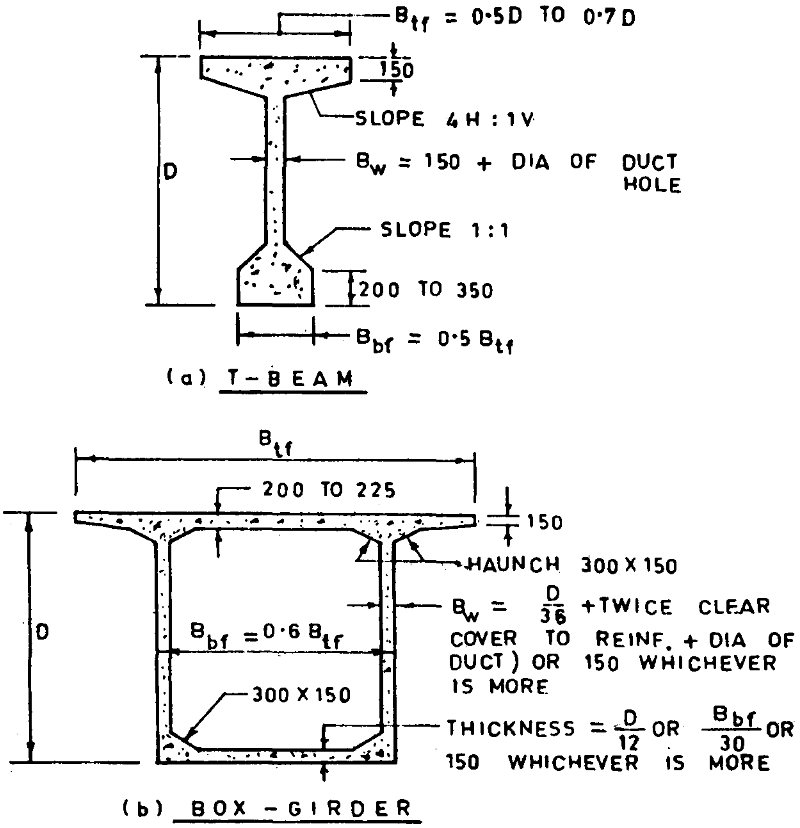


FIG. 16·6 PRELIMINARY GIRDER DIMENSIONS

The approximate depth of girders for prestressed concrete decks may be determined from the following to start with the preliminary design to meet the requirements of IRC : 18-1985 (L and D are span and depth of girders in metres).

a) ***T-beam and slab bridges (7.5 m. carriage way)***

i) For 3 beams deck, $D = \frac{L}{16}$.

ii) For 4 beams deck, $D = \frac{L}{18}$.

iii) For 5 beams decks, $D = \frac{L}{20}$.

b) ***Box-girder bridges***

i) For single cell deck, $D = \frac{L}{16}$.

ii) For twin cells deck, $D = \frac{L}{18}$.

iii) For three cells deck, $D = \frac{L}{20}$.

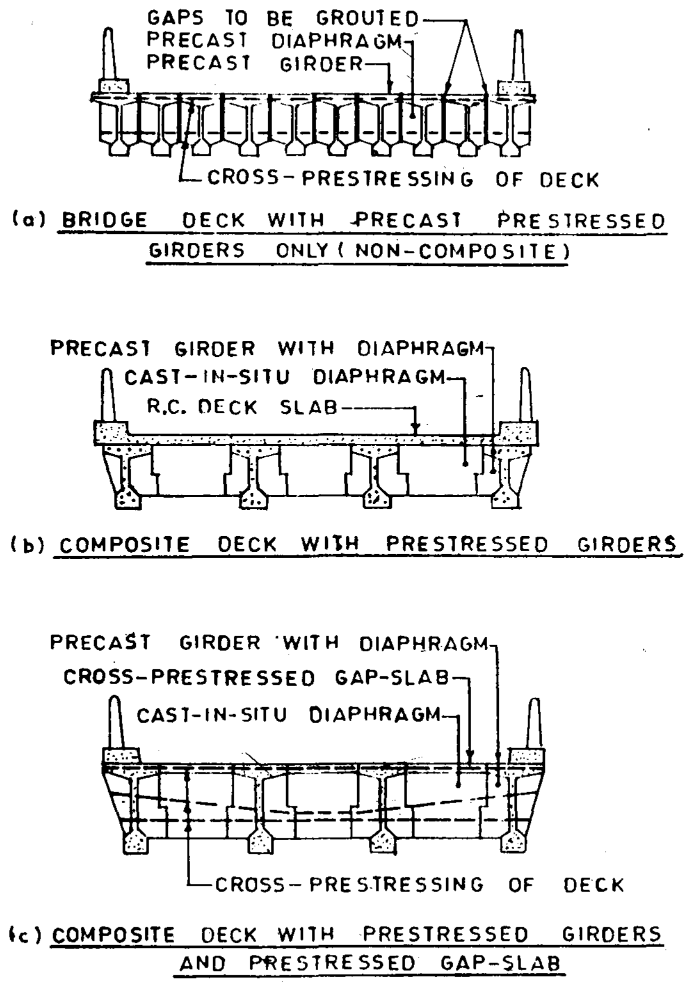


FIG. 16·7 PRESTRESSED CONCRETE T-BEAM DECKS

16.7 H.T. CABLE (ARPROX. NOS.) (To meet the requirements of IRC : 18-1985)

Total Nos. of high tensile cables (12 wires of 7 mm. dia) may be assumed in the preliminary design as 1.6 to 1.7 times the span in metres. For a 45 m simply supported deck with 5 Nos. beams, total nos. of cables required as per thumb rule are $45.0 \times 1.7 = 76.5$. Nos. of cables actually used are 15 Nos. (average) per girder. In a box-girder bridge with contilever construction having a span of 101.0 m. Nos. of cables as per thumb rule come to $1.7 \times 101 = 171.7$. Nos. of cables actually used = 172 Nos.

16.8 DESIGN PRINCIPLES

16.8.1 In non-composite decks, the girders are placed side by side with a gap of 25 to 40 mm. in between the flanges and the diaphragms, Fig. 16.7a. This type of decks is usually adopted where the head-room is restricted or the launching of the girders is essential due to difficulty in centering work. The girders are precast in casting yard, prestressed and then launched in position by some device. The joints are then grouted with cement-sand grout and the deck is prestressed transversely so as to make it rigid and monolithic.

16.8.2 In composite decks, on the other hand, the girders may be cast at site or precast at casting yard and launched after initial prestressing. R. C. slab over the prestressed girders and R. C. diaphragms are cast and made composite with the help of shear connectors. This type of deck is shown in Fig. 16.7b. Another type of prestressed concrete composite deck as illustrated in Fig. 16.7c is also used. In such decks, gap slabs and gap diaphragms are cast after the girders are launched in position and the deck and the diaphragms are cross-prestressed.

16.8.3 In the type of decks illustrated in Fig. 16.7a, since the sectional properties such as areas, section modulii etc. remain unchanged for all conditions of loading, the stresses in the girders are worked out with the same sectional properties throughout. In composite decks, however, the section properties of the girders are changed after the deck slab or the gap slab is made composite with the girders and as such in calculating the stresses, modified properties of the composite girders are to be taken into account. This means that the stresses due to self weight of the girders, first stage of prestressing, weight of deck or gap slab etc. are to be calculated with the non-composite girder section only when the girders are not propped but after the casting and the attainment of the necessary strength in the deck slab, the stresses due to succeeding stages of prestressing, weight of wearing course, railing etc. and those due to live load are to be worked out on the basis of composite sectional properties which are greater than the non-composite ones.

16.8.4 Prestressing is generally done in two or three stages in composite decks in order to reduce the effect of the secondary dead load such as deck slab, wearing course etc. as well as to reduce the losses due to creep and shrinkage as far as possible. This is an advantage of the composite decks over the non-composite ones.

16.8.5 Kern Distances

16.8.5.1 For non-composite girders, the area of cross-section, A and the section Modulii Z_t and Z_b of the section will remain the same at the initial as well as at the final (service) stage. Therefore, if P is the prestressing force, M_D is the moment due to dead loads and M_L is the moment due to live load, then the stresses at top and bottom of girder viz. 6_t and 6_b are given by the following equations (see also Fig. 16.8).

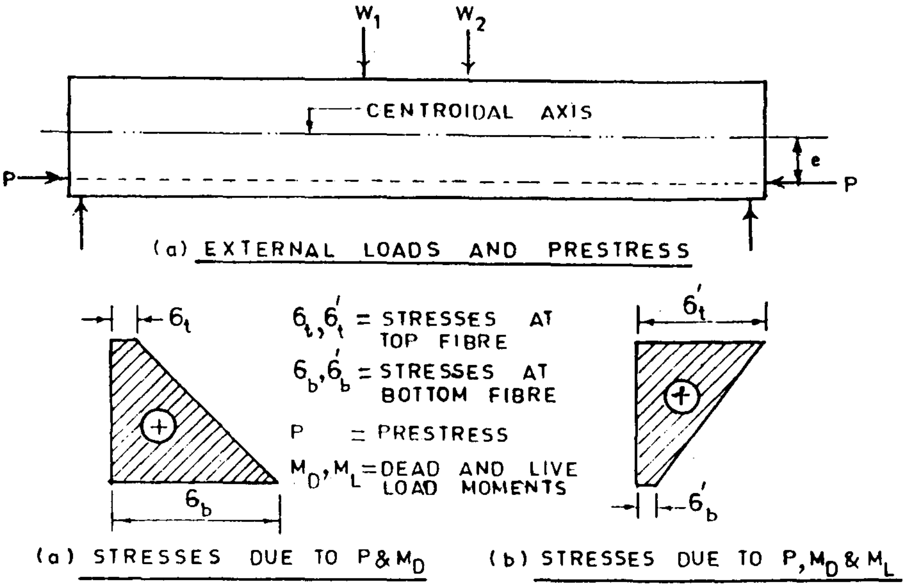


FIG 16·8 STRESSES DUE TO P, M_D AND M_L

i) Due to prestress (with an eccentricity, e) and M_D

$$\sigma_t = \frac{P}{A} - \frac{P \cdot e}{Z_t} + \frac{M_D}{Z_t} \tag{16.4}$$

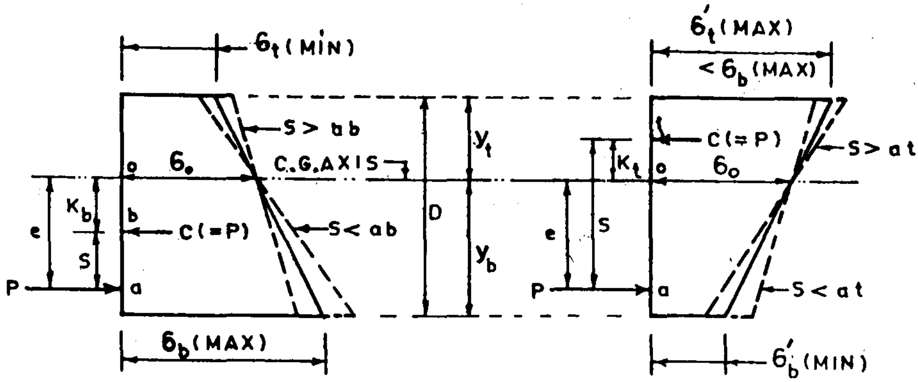
$$\sigma_b = \frac{P}{A} + \frac{P \cdot e}{Z_b} - \frac{M_D}{Z_b} \tag{16.5}$$

ii) Due to prestress (with an eccentricity, e), M_D and M_L

$$\sigma'_t = \frac{P}{A} - \frac{P \cdot e}{Z_t} + \frac{(M_D + M_L)}{Z_t} \tag{16.6}$$

$$\sigma'_b = \frac{P}{A} + \frac{P \cdot e}{Z_b} - \frac{(M_D + M_L)}{Z_b} \tag{16.7}$$

16.8.5.2 The presssure line i.e. the resultant of the compressive stresses induced by the prestress force coincides with the prestress profile when external loads are not acting on the beam. The pressure line shifts with the application of external loads to provide the lever arm required for the resisting couple. These are shown in Fig. 16.9.



(a) STRESSES DUE TO P AND M_D

(b) STRESSES DUE TO P M_D AND M_L

FIG.16·9 SHIFTING OF THE PRESSURE LINE OF PRESTRESSED BEAMS (Mallick et al)[9]

From Fig. 16.9a,

$$\sigma_b = \frac{P}{A} + \frac{P(ob)}{Z_b} \tag{16.8}$$

and, $$\sigma_t = \frac{P}{A} - \frac{P(ob)}{Z_t} \tag{16.9}$$

But $P = \sigma_o A$ and $I = Ar^2$

Hence from Equation 16.8; $\sigma_b = \frac{\sigma_o A}{A} + \frac{\sigma_o A \cdot ob}{Ar^2} \cdot y_b = \sigma_o \left[1 + \frac{ob \cdot y_b}{r^2} \right]$

$$\text{or,} \quad ob = \frac{r^2}{y_b}\left[\frac{6_b}{6_o} - 1\right] \tag{16.10}$$

Also from equation 16.9,

$$ob = K_b = \frac{r^2}{y_t}\left[1 - \frac{6_t}{6_o}\right] \tag{16.11}$$

16.8.5.3 The two values are equal if $6_o = \left[\frac{6_b \cdot y_t + 6_t \cdot y_b}{D}\right]$. The ordinate *ab* is the shift of the pressure line under the dead load moment M_D and if C does not move upto *b* i.e. the shift, $S = \frac{M_D}{P} <$ ab but if C moves beyond b (towards 0) then the shifts $S <= \frac{M_D}{P} >$ ab. Stress distributions under these conditions are shown in Fig. 16.9a. Stress at bottom fibre under dead load and prestress should not exceed 6_b (max) and stress at top fibre under dead load and prestress should be as close as possible to 6_t (min). This condition is satisfied when S = ab. The distance *ob* denoted by K_b is known as the *"bottom or lower kern"* distance which is given by,

$$S = e - K_b \tag{16.12}$$

$$\text{or} \quad P(e - K_b) = M_D \tag{16.13}$$

$$\text{or} \quad e = \frac{M_D}{P} + K_b \tag{16.14}$$

16.8.5.4 Similarly, the stress distribution under prestress, dead load and live load are shown in Fig. 16.9b. Under these loading condition, the pressure line is shifted to *t*. The ordinate *ot* is termed as the *"top or upper kern"* distance.

$$\text{As before,} \quad ot = K_t = \frac{r^2}{y_b}\left[1 - \frac{6'_b}{6_o}\right] \tag{16.15}$$

$$\text{Also} \quad ot = K_t = \frac{r^2}{y_b}\left[\frac{6'_t}{6_o} - 1\right] \tag{16.16}$$

and the two values of K_t are equal.

$$S = e + K_t \tag{16.17}$$

$$P(e + K_t) = M_D + M_L \tag{16.18}$$

$$\text{or,} \quad e = \left[\frac{(M_D + M_L)}{P} - K_t\right] \tag{16.19}$$

From equations 16.13 and 16.18,

$$P(K_b + K_t) = ML \tag{16.20}$$

16.8.5.5 Since the minimum stress governs the design, the kern distances K_b and K_t are given by equations 16.11 and 16.15, which are as below :

$$K_b = \frac{r^2}{y_t}\left[1 - \frac{6_t\,(\text{min})}{6_o}\right] \tag{16.21}$$

$$K_t = \frac{r^2}{y_b}\left[1 - \frac{6'_b\,(\text{min})}{6_o}\right] \tag{16.22}$$

The profile of the resultant prestress along the length of the beam may be obtained from the loci of the kern distances considering the variation of the bending moment alongwith span. In consideration of the above, the resultant prestress profile shall be located within the zone given by,

$$e\ (\text{max.}) = \left[\frac{M_D}{P} + K_b\right] \quad \text{for dead load, and} \tag{16.23}$$

$$e\ (\text{min}) = \left[\frac{(M_D + M_L)}{P} - K_t\right] \quad \text{for dead and live load} \tag{16.24}$$

16.8.5.6 The limiting zone for a simply supported beam under uniformly distributed load is shown in Fig. 16.10. The limiting zone is enclosed by the curves for $\frac{M_D}{P}$ and $\frac{(M_D + M_L)}{P}$ and measured downwards from the lines *b-b* and *t-t* respectively. The obligatory point for the passage of the prestress profile is obtained when *a* and *c* coincide. The point *a* will be below *c* when the section is inadequate but above *c* when the section is oversized.

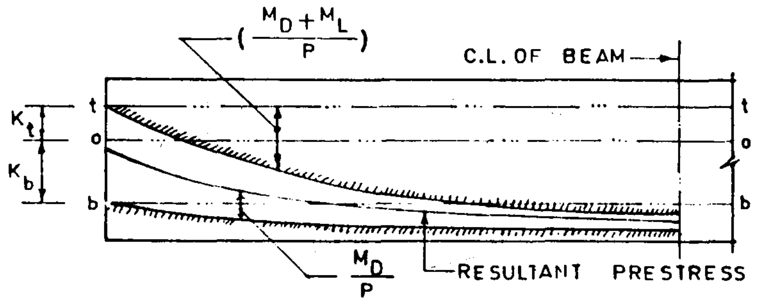


FIG.16·10 LIMIT ZONE FOR RESULTANT PRESTRESS FOR A SIMPLY SUPPORTED BEAM (Mallick et al)[9]

16.8.5.7 Approximate Kern Distances

The kern distances have important role in the selection of the sections and as such an approximate method for determination of the kern distances is given below :

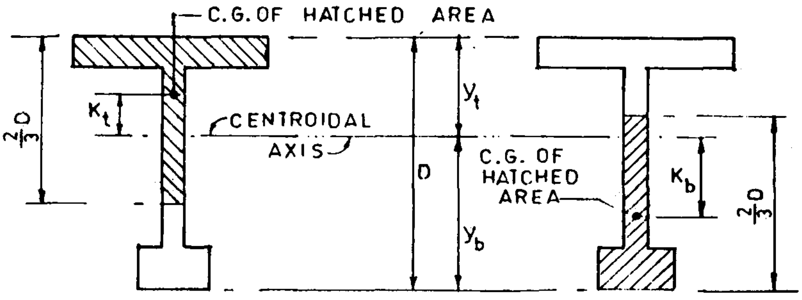


FIG. 16·11 APPROXIMATE KERN DISTANCES (Mallick et al)[9]

The minimum stress σ_t (min) in Fig. 16.9a and σ_b' (min) in Fig. 16.9b may be assumed as zero without appreciable error. For this condition of triangular stress distribution, the centre of gravities of the hatched areas in Fig. 16.11a and 16.11b may be considered as top and bottom kerns approximately.

16.8.6 Design of Section

The adequacy of the prestressed concrete girder section should be checked in respect of the following :

16.8.6.1 *Stress during erection and at service*

The stresses at top and bottom fibres due to the action of dead loads, prestress and the live loads should remain within the permissible limits. The moments produced due to dead load, live load and the eccentricity of the prestressing force are to be considered for this. The sections at 1/4, 1/3, 3/8 and 1/2 span should also satisfy the design requirements. The cable profile requires to be fixed accordingly.

16.8.6.2 *Ultimate strength for bending*

The girders should also be checked for their ultimate strength. For this purpose, the ultimate moments of resistance of the girders as well as the ultimate moments that may be produced due to certain over loading may also have to be worked out and compared. The girders are to be checked for the following ultimate loads :

i) Ultimate load = 1.25G + 2.0 SG + 2.5 Q (16.23)

under normal exposure condition.

ii) Ultimate load = 1.5 G + 2.0 SG + 2.5 Q (16.24)

under severe exposure condition

iii) Ultimate load = G + SG + 2.5 Q (16.25)

where dead load causes effects opposite to those of live load.

In the above expressions, G, SG and Q are permanent load, superimposed dead load (such as dead load of precast footpath, hand-rails, wearing course, utility services etc.) and live loads including impact respectively.

The ultimate moments of resistance for concrete or steel are given by :

i) M_u of concrete $= 0.176\, bd^2\, fck$ for rectangular section (16.26)

ii) M_u of concrete $= 0.176\, bd^2\, fck + \frac{2}{3} \times 0.8\,(B_f - b)\,(d - \frac{t}{2})\, t \cdot fck$ for a T Section. (16.27)

iii) M_u of steel $= 0.9\, d\, As\, f_p$ (16.28)

Where
- b = The width of rectangular section or web of T-beam.
- d = Effective depth of beam from C. G. of H. T. Steel
- f_{ck} = Characteristic strength of concrete.
- B_f = The width of flange of T-beam.
- t = The thickness of flange of a T-beam.
- A_s = The area of high tensile steel.
- f_p = The ultimate tensile strength of steel without definite yield point or yield stress or stress at 4 percent elongation whichever is higher for steel with a definite yield point.

The section shall be so proportioned that M_u for steel is less than that for concrete so that failure may occur by yielding of steel rather than crushing of concrete.

16.8.6.3 *Shear*

i) The checking of shear shall be made for ultimate load. The ultimate shear resistance of the concrete, V_C at any section shall be evaluated both for uncracked and cracked section in flexure and the lesser value shall be taken and shear reinforcement provided accordingly.

ii) The ultimate shear resistance of uncracked section,

$$V_{cu} = 0.67\, bD \sqrt{f_t^2 + 0.8\, f_{cp} \cdot f_t} \tag{16.29}$$

Where b = the width of rectangular section or the width of rib for T, I or L-beam.
D = overall depth of the member
f_t = maximum principal stress given by $0.24\sqrt{fck}$
f_{cp} = compressive stress at centroidal axis due to prestress taken as positive.

The component of the prestressing force normal to the longitudinal axis of the member may be added to V_{cu}.

iii) The ultimate shear resistance of cracked section,

$$V_{cc} = 0.037\, bd \sqrt{fck} + \frac{Mt}{M} \cdot V \;\; \text{but} < 0.1\, bd \sqrt{fck} \tag{16.30}$$

Where d = Effective depth from the C. G. of steel tendon
Mt = the cracking moment at the section = $(0.37\sqrt{fck} + 0.8\, f_{pt})\dfrac{I}{y}$ in which f_{pt} is the stress due to prestress only at the tensile fibre distance y from the centroid of the concrete section having a second moment of area, I.
V & M = shear force and the corresponding bending moment at the section due to ultimate load.

The component of the prestressing force normal to the longitudinal axis may be ignored.

iv) *Shear Reinforcement*

When V, the shear force due to ultimate load is less than $\frac{V_c}{2}$ (where V_c is the lesser of V_{cu} or V_{cc} as given above), then no shear reinforcement is necessary. When V is greater than $\frac{V_c}{2}$, a minimum shear reinforcement in the form of links shall be provided as below :

$$\frac{Asv}{Sv} \times \frac{0.87\, fy}{b} = 0.4 \text{ MPa} \tag{16.31}$$

When the shear force V, exceeds V_c, shear reinforcement shall be provided as under :

$$\frac{Asv}{Sv} = \frac{(V - V_c)}{0.87\, fy\, d} \tag{16.32}$$

Where Asv = the cross-sectional area of the two legs of a link
Sv = the spacing of the links
fy = the yield strength or 0.2 percent proof stress of the reinforcement but not greater than 415 MPa.

Vc = the shear force carried by the concrete section.
d = the depth of the section from the extreme compression fibre either to the longitudinal bars or to the centroid of the tendons whichever is greater.

v) *Maximum Shear Force*

The shear force V due to ultimate loads shall not exceed τ_c bd, the values of τ_c being given in Table 16.6.

TABLE 16.6 : VALUE OF τ_c (MAX. SHEAR STRESS) *(IRC)*[1]

Concrete grade	M30	M35	M40	M50	M50 and over
Max. shear stress (MPa)	4.1	4.4	4.7	5.3	5.5
Note : Intermediate values may be obtained by linear interpolation.					

16.8.6.4 *Torsion*

The effect of torsion is generally less and the nominal shear reinforcement provided is normally adequate to resist the torsional stress. Where torsional resistance or stiffness of the members is taken into consideration in the analysis of the structure, check for torsion and additional reinforcement to resist torsion are necessary. The details given in Clauses 14.2.2 and 14.2.3 of IRC : 18-1985 may be followed in such cases.

16.9 MINIMUM REINFORCEMENT

16.9.1 Minimum reinforcement in the vertical direction to be provided in the bulb/web of beams/rib of box-girders shall be not less than 0.3 percent for mild steel bars or 0.18 percent for HYSD bars of the cross-sectional area in plan.

16.9.2 Minimum longitudinal reinforcement shall be not less than 0.25 percent for mild steel bars or 0.15 percent for HYSD bars of the cross-sectional area in elevation for grade of concrete less than M45. The percentages of reinforcement for mild steel or HYSD bars shall be increased to 0.3 or 0.18 in case grade of concrete exceeds M45.

16.9.3 The diameter of bars as required in Art. 16.9.1 and 16.9.2 shall not be less than 10 mm. for m.s. bars or 8 mm. for HYSD bars the spacing of such bars shall not be more than 200 mm.

16.9.4 The minimum reinforcement for solid slab decks or top slab of box girders shall be not less than 0.3 percent for M.S. bars and 0.18 for percent for HYSD bars. The same for soffit slab of box girders shall not be less than 0.3 percent for M.S. bars or 0.18 percent for HYSD bars in the longitudinal direction and 0.5 percent for M.S. bars or 0.3 percent for HYSD bars in the transverse direction. Such percentage of reinforcement shall be determined on the basis of cross-sectional area in each direction and the reinforcement be placed equally at top and bottom.

16.9.5 Minimum reinforcement for cantilever slab shall be 6 nos. of 16 mm. dia. M.S. bars or 4 nos. 16 mm. dia HYSD bars and be placed equally at top and bottom parallel to the support at the tip of the slab with minimum spacing.

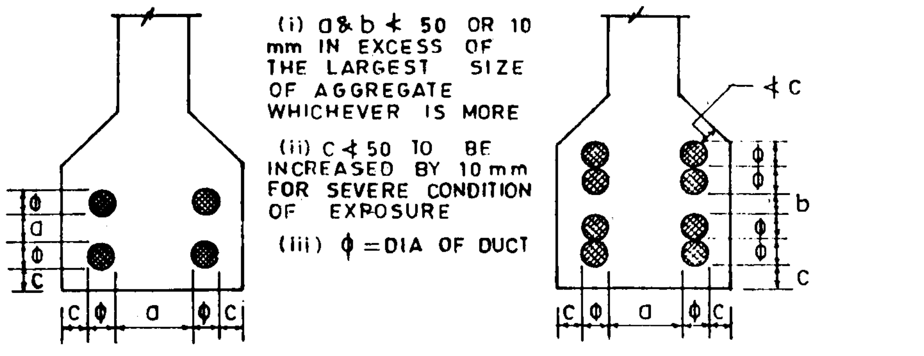
16.10 COVER AND SPACING OF PRESTRESSING STEEL

16.10.1 IRC : 18-1985 specifies that clear cover to untensioned reinforcement including links and stirrups shall be as indicated in Table 16.7. IRC : SP-33, however, recommends that for important bridges, the minimum clear cover shall be 50 mm. but the same shall be increased to 75 mm. wherever the prestressing cable is nearest to the concrete surface.

TABLE 16.7 : CLEAR COVER TO UNTENSIONED REINFORCEMENT *(IRC)* [1]

Grade of concrete	Conditions of exposure	Nominal cover in mm.
Less than M40	Moderate	30
	Severe	40
M40 and more	Moderate	25
	Severe	30

16.10.2 As specified in IRC : 18-1985, clear cover measured from the outside of the sheathing, spacing and grouping of cables shall be as indicated in Fig. 16.12. However, for important bridges, the recommendation of IRC : SP-33 is that a clear spacing of 100 mm. shall be provided for cables or group of cables to be grouted later. SP-33 also recommends that for segmental construction where multi-stage prestressing is adopted, clear spacing shall not be less than 150 mm. between the first and subsequent groups of cables.



(a) FOR SINGLE CABLES (b) FOR GROUPED CABLES

FIG.16·12 COVER, SPACING AND GROUPING OF CABLES(IRC)[1]

16.11 END BLOCKS

16.11.1 To cater for heavy stresses at the anchorage ends, some length of the web at the ends is thickened which is known as *"End Block"*. Generally, the web at the ends is made as wide as the bottom bulb and this is achieved gradually from the normal width by providing a splay of 1 in 4. The length of end blocks shall be at least equal to its width or 600 mm. whichever is more. The end block shall have sufficient area to accommodate all the anchorages at the jacking ends. It is preferable to precast the portion of the end block housing the anchorages. The design of the end blocks shall be based on the provisions made in Clause 17.2 and 17.3 or IRC : 18-1985.

ILLUSTRATIVE EXAMPLE 16.1

Design the mid-span section of the central girder of a 3-beams bridge deck of the type shown in Fig. 16.7b.

Given

i)	*Effective span of the girder*	*39.0 m.*
ii)	*Carriageway·*	*7.5 m.*
iii)	*Footpath*	*1.5 m. on either side*
iv)	*Loading : Single lane of IRC 70R or 2 lanes of IRC Class A*	
v)	*Spacing of girders*	*2.7 m.*
vi)	*Thickness of deck slab*	*200 mm.*
vii)	*Grade of concrete : M 42.5 for PSC girders and M30 for deck slab.*	
viii)	*Live load distribution coefficient : 1.10 both for two lanes of Class A and single lane of Class 70R loading.*	

Solution

New types of calculation are only shown here; calculations of the type done before in the preceding chapters are not shown here for which only abstracts are shown. The design is based on IRC : 18-1985 (since this code is still in vogue) to illustrate the design principles of a prestressed concrete bridge deck. The provisions in respect of depth of girder, thickness of web, spacing of cables, anchorage of cables in the top deck surface, stages of prestressing, maximum jacking force, permissible stresses during service in consideration of differential temperature etc. require to be modified if the design has to be done on the recommendations of IRC : SP 33-1988.

Step 1 Preliminary dimensions of the girder

From Art. 16.6

i) D for 3 beam deck $= \frac{L}{16} = \frac{39.0}{16} = 2.44$ m. say 2.5 m.

ii) Btf = 0.5 to 0.7 D i.e. 1.25 to 1.75 m. Let us adopt Btf = 1.25 m.

iii) Bbf = 0.5 Btf = 0.5 $\times$ 1.25 = 0.625 m. Say 0.6 m.

iv) Bw = 150 mm + dia. of duct hole = 150 + 40 = 190 mm.

The details of the bridge deck and the girders are shown in Fig. 16.13 and 16.14 respectively.

TABLE 16.8 : CALCULATION OF AREA, CG OF SECTION, y_t AND y_b (REF. FIG. 16.14)

Item of girder	A = Area (cm^2)	x=CG of area from bottom (cm)	Ax (cm^3)	y_b = CG of the girder section from bottom (cm)
1	$60 \times 30 = 1800$	15.0	27,000	$y_b = \frac{\Sigma Ax}{\Sigma A} = \frac{11,87,000}{8,690} = 137$ cm $y_t = 250 - 137 = 113$ cm.
2	$\frac{1}{2} \times 2 \times 20 \times 20.5 = 420$	36.67	15,400	
3	$19 \times 205 = 3895$	132.50	5,16,090	
4	$125 \times 15 = 1875$	242.5	4,54,690	

TABLE 16.8 : CALCULATION OF AREA, CG OF SECTION, y_t AND y_b (REF. FIG. 16.14) *Contd.*

Item of girder	A = Area (cm^2)	x=CG of area from bottom (cm)	Ax (cm^3)	y_b = CG of the girder section from bottom (cm)
5	$2 \times \frac{1}{2} \times 31 \times 5 = 155$	233.0	36,115	
6	$2 \times \frac{1}{2} \times 22.5 \times 22.5 = 506$	222.5	1,12,580	
7	$2 \times 22.5 \times 5 = 225$	232.5	52,310	
	$\Sigma A = 8686$ Say 8690 cm^2	$\Sigma Ax = 11,87,005$ Say 11,87,000 cm^3		

TABLE 16.9 : MOMENT OF INERTIA AND SECTION MODULII OF GIRDER SECTION (REF. FIG. 16.14)

Item of girder	I_o (own axis) (cm^4)	$\bar{x}$ = CG distance of each area from CG of Girder (cm)	$A\bar{x}^2$ (cm^4)	$I = I_o + A\bar{x}^2$ (cm^4)
1	$\frac{60 \times 30^3}{12} = 1,35,000$	(137 – 15) = 122	2,67,91,200	2,69,26,200
2	$\frac{2 \times 20 \times 20.5^3}{36} = 56,000$	(137 – 36.67) = 100.33	42,27,800	42,83,800
3	$\frac{19 \times 205^3}{12} = 1,29,22,700$	(137 – 132.50) = 4.5	78,900	1,30,01,600
4	$\frac{125 \times 15^3}{12} = 35,100$	(242.5 – 137) = 105.5	2,08,69,200	2,09,04,300
5	$\frac{2 \times 31 \times 5^3}{36} = 200$	(233 – 137) = 96	14,28,500	14,28,700
6	$\frac{2 \times 22.5 \times 22.5^3}{36} = 5,12,600$	(222.5 – 137) = 85.5	36,99,000	42,11,600
7	$\frac{2 \times 22.5 \times 5^3}{12} = 400$	(232.5 – 137) = 95.5	20,52,100	20,52,500
				$\Sigma I = 7,28,08,700$

$$\therefore \quad Z_t = \frac{\Sigma I}{y_t} = \frac{7,28,08,700}{113} = 6,44,300 \text{ cm}^3 \; ; \; Z_b = \frac{\Sigma I}{y_b} = \frac{7,28,08,700}{137} = 5,31,400 \text{ cm}^3$$

Step 2 Self weight of PSC girder

Area of cross-section of the girder from Table 16.8 = 0.869 m^2

$\therefore$ Weight per m = $0.869 \times 1.0 \times 2500 = 2170$ kg.

Weight of intermediate stiffener (5 Nos.) 200 mm thick = 1040 Kg.

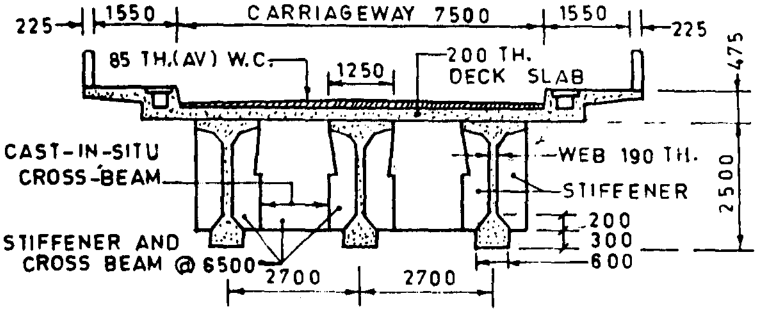


FIG.16·13 DETAILS OF BRIDGE DECK (Example 16·1)

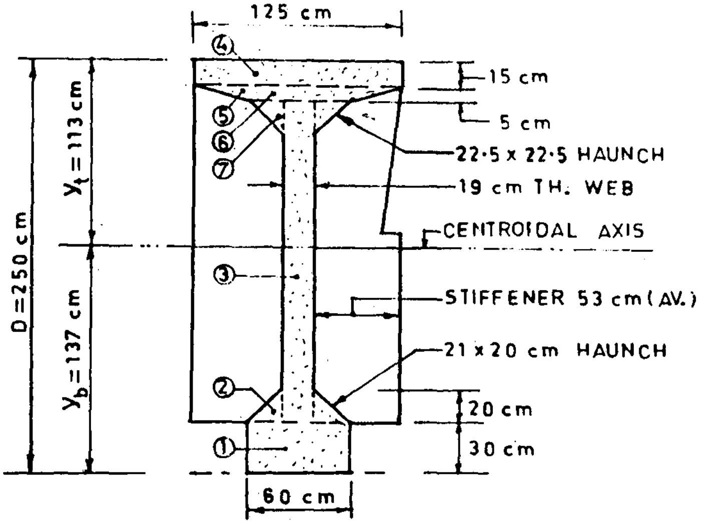


FIG 16·14 DETAILS OF PSC GIRDER (Example 16·1)

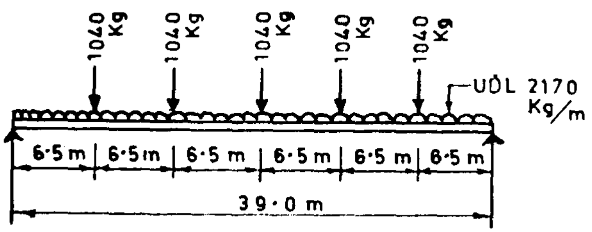


FIG.16·15 LOAD ON GIRDER (Self weight plus wt. of stiffener) (Ex. 16·1)

Step 3 Bending moment due to self weight of girder

$$R_A = \frac{1}{2}(39.0 \times 2170 + 5 \times 1040) = 44{,}915 \text{ Kg.}$$

M_D = moment at mid-span = $44{,}915 \times 19.5 - 2170 \times 19.5 \times \frac{19.5}{2} - 1040(13.0 + 6.5) = 4{,}43{,}000$ Kgm.

Stress due to self weight of girder at top and bottom :

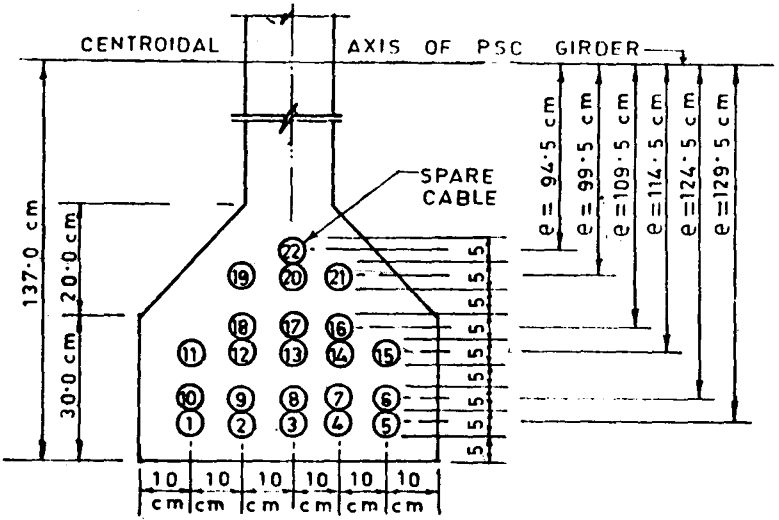
$$\sigma_{tg} = \frac{M_D}{Z_t} = \frac{4{,}43{,}000 \times 100}{6{,}44{,}300} \ (+)\ 68.76\ \text{Kg/cm}^2$$

$$\sigma_{bg} = \frac{M_D}{Z_b} = \frac{4{,}43{,}000 \times 100}{5{,}31{,}400} \ (-)\ 88.36\ \text{Kg/cm}^2$$

Step 4 Prestress

Using high tensile cables consisting of 12 nos. 7 mm. dia wires, nos. required as per thumb rule are given in Art. 16.7. Let us adopt the lower value of the multiplying factor since the depth adopted is slightly more (2.5 m in place of 2.44 m) in the preliminary design. ∴ Total Nos. of cables = 1.6 × 39.2 = 62. Adopt 21 cables per girder as a first trial.

Step 5 Arrangement of cables at mid span



(i) ALL CABLES ARE OF 12 NOS 7 ϕ
(ii) CABLES MKD 1,2,3,4,5,7 & 9 ARE FIRST STAGE CABLES
(iii) CABLES MKD 6,8,10,11,12,14 & 15 ARE SECOND STAGE CABLES
(iv) CABLES MKD 13,16,17,18,19,20 & 21 ARE THIRD STAGE CABLES
(v) CABLE MKD 22 IS A SPARE CABLE

FIG. 16·16 ARRANGEMENT OF CABLES AT MIDSPAN

The arrangement of cables at mid span of the girder is shown in Fig. 16.16 in line with the provisions of Art. 16.10.2.

Step 6 Stages of Prestressing

The prestressing is proposed to be done in 3 stages as below :

1st Stage 7 Cables viz. no. 1 to 5 and 7, 9 will be stressed after 10 days of casting of girder when the concrete strength will be about 75%.

2nd Stage 7 cables viz. nos. 6, 8, 10, 11, 12, 14, 15 will be stressed after 28 days of casting of girder. Thereafter, the girder will be launched in position, cross girders and deck slab cast. After 28 days of deck slab casting, road kerb, footway slab railing etc. will be done.

3rd Stage 7 cables viz. no. 13 and 16 to 21 will be stressed after 90 days of casting of girders and then wearing course will be laid.

Step 7 Stresses in Cables

i) Ultimate Tensile strength of cables = 158 Kg/mm^2

ii) Stress at midspan for 1st and 2nd stage cable = 56% of UTS = 88.5 kg/mm^2

iii) Stress at midspan for 3rd stage (length being less, friction is less) = 60% of UTS = 94.8 kg/mm^2

iv) Prestressing force for each 1st and 2nd stage cables :

$$= 88.5 \times 12 \times \frac{\pi}{4} \times (7.0)^2 = 40.86 \times 10^3 \text{ Kg.}$$

v) Prestressing force for each 3rd stage cables :

$$= 94.8 \times 12 \times \frac{\pi}{4} \times (7.0)^2 = 43.78 \times 10^3 \text{ Kg.}$$

TABLE 16.10 : PRESTRESSING FORCE AND ECCENTRICITY FOR FIRST & SECOND STAGE CABLES

Stage	Cable No.	Eccentricity, e (cm)	Force per cable (Kg)	Total force P (Kg)
1st	1, 2, 3, 4, 5, 7, 9 = 7 Nos.	For 5 cables = 129.5 For 2 cables = 124.5	40.86×10^3	$7 \times 40.86 \times 10^3$ $= 286.02 \times 10^3$
2nd	6, 8, 10, 11, 12, 14, 15 = 7 Nos.	For 3 cables = 124.5 For 4 cables = 114.5	40.86×10^3	$7 \times 40.86 \times 10^3$ $= 286.02 \times 10^3$

Step 8 Moments due to Prestress

i) Due to 1st stage cables :

$$M_p = 5 \times 40.86 \times 10^3 \times 129.5 + 2 \times 40.86 \times 10^3 \times 124.5$$

$$= (26{,}457 + 10{,}174) \times 10^3 = 36{,}631 \times 10^3 \text{ Kg.cm.}$$

ii) Due to 2nd stage cables :

$$M_p = 3 \times 40.86 \times 10^3 \times 124.5 + 4 \times 40.86 \times 10^3 \times 114.5$$

$$= (15{,}261 + 18{,}714) \times 10^3 = 33{,}975 \times 10^3 \text{ Kg.cm.}$$

Step 9 Stresses due to Prestress

i) 1st stage prestress (Ref. Table 16.8 to 16.10)

Stress at top of girder, $6_{tg} = \frac{P}{A} - \frac{M_P}{Z_t} = \frac{286.02 \times 10^3}{8690} - \frac{36{,}631 \times 10^3}{6{,}44{,}300}$

$= (+)\ 32.91 - 56.85 = (-)\ 23.94$ Kg/cm²

Stress at bottom of girder, $6_{bg} = \frac{P}{A} + \frac{M_P}{Z_b} = \frac{286.02 \times 10^3}{8690} + \frac{36{,}631 \times 10^3}{5{,}31{,}400}$

$= (+)\ 32.91 + 68.93 = (+)\ 101.84$ Kg/cm²

ii) 2nd stage prestress (Ref. Table 16.8 to 16.10)

$$6_{tg} = \frac{286.02 \times 10^3}{8690} - \frac{33{,}975 \times 10^3}{6{,}44{,}300} = (+)\ 32.91 - 52.73 = (-)\ 19.82 \text{ Kg/cm}^2$$

$$6_{bg} = \frac{286.02 \times 10^3}{8690} + \frac{33{,}975 \times 10^3}{5{,}31{,}400} = (+)\ 32.91 + 63.98 = (+)\ 96.89 \text{ Kg/cm}^2$$

Step 10 Stresses at top and bottom of girder due to self-weight of girder and 1st stage prestress.. [Step 3 + Step 9(i)]

$6_{tg} = (+)\ 68.76 - 23.94 = (+)\ 44.82$ Kg/cm² $6_{bg} = (-)\ 83.36 + 101.84 = (+)\ 18.48$ Kg/cm²

Step 11 Partial loss of prestress of 1st stage cables at the time of 2nd stage prestressing

2nd stage prestressing is done at 28 days after casting of girder and at 18 days after 1st stage of prestress.

i) ***Loss due to creep***

From Table 16.2, creep strain at 75% maturity of concrete at 10 days is 5.6×10^{-4} and that at 100% maturity at 28 days is 4.0×10^{-4} per 10 MPa stress.

Therefore, differential creep strain at 18 days is $(5.6 - 4.0) \times 10^{-4}$ per 10 MPa stress. Concrete stress at the bottom fiber after 1st stage prestress = 18.48 kg/cm² (Refer Step 10).

$\therefore$ Stress at the C.G. of 1st stage cables $= 18.48 \times \frac{(137 - 7.5 - 7.14)}{137} = 16.50$ Kg/cm² = 1.7 MPa

Hence net differential creep strain $= (5.6 - 4.0) \times 10^{-4} \times \frac{1.7}{10} = 0.27 \times 10^{-4}$

From Table 16.1, $E_s = 2.1 \times 10^5$ MPa

Loss of stress = Es × Strain $= 2.1 \times 10^5 \times 0.27 \times 10^{-4} = 5.67$ MPa = 0.57 Kg/mm²

Percentage of loss of prestress $= \frac{0.57}{88.5} \times 100 = 0.64$

ii) ***Loss due to shrinkage of concrete*** (Ref. Table 16.3)

Residual shrinkage strain at 10 days $= 3.0 \times 10^{-4}$

Residual shrinkage at 28 days $= 1.9 \times 10^{-4}$

Shrinkage strain during this period of 18 days $= (3.0 - 1.9) \times 10^{-4} = 1.1 \times 10^{-4}$

$\therefore$ Loss of stress due to shrinkage = Es × shrinkage strain

$= 2.1 \times 10^5 \times 1.1 \times 10^{-4} = 23.1$ MPa $= 2.36$ Kg/mm^2

$\therefore$ Percentage loss $= \dfrac{2.36}{88.5} \times 100 = 2.67$

iii) ***Loss due to relaxation of steel*** (Art. 16.5.4)

Relaxation loss for initial prestress of 0.56 of UTS for 1st stage cable (Refer Step 7)
$= 21$ MPa $= 2.14$ Kg/mm^2

$\therefore$ Percentage loss $= \dfrac{2.14}{88.5} \times 100 = 2.42$

$\therefore$ Total loss for (i) to (iii) $= 0.64 + 2.67 + 2.42 = 5.73\%$

Step 12 Stresses at top and bottom of girder due to self weight of girder and 1st stage prestress after loss of 5.73% before 2nd stage prestress.

$6_{tg} = (+)\ 68.76 - 23.9 \times 0.943 = (+)\ 46.18$ Kg/cm^2 $\quad 6_{bg} = (-)\ 83.36 + 101.84 \times 0.943 = (+)\ 12.68$ Kg/cm^2

Step 13 Stresses in girder due to self wt. of girder, + 1st stage prestress with loss + 2nd stage prestress at 28 days [Step 12 and 9(ii)]

$6_{tg} = (+)\ 46.18 - 19.82 = (+)\ 26.36$ Kg/cm^2 $\quad 6_{bg} = (+)\ 12.68 + 96.89 = (+)\ 109.57$ Kg/cm^2

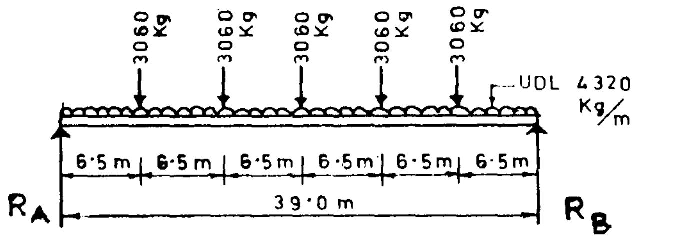


FIG. 16·17 LOAD ON GIRDER (Weight of deck slab and cross beam) (Example 16·1)

Permissible temporary stresses as per Art. 5.21.2.2 of Chapter 5 = 0.45 f_{ck} but not more than 20 MPa; i.e. 0.45 $\times$ 42.5 = 19.12 MPa = 195 Kg/cm^2 (compression) and 0.1 $\times$ 195 = 19.5 Kg/cm^2 (tension). Therefore, after the transfer of 2nd stage of prestressing, the stresses in the girder are within permissible limits.

Step 14 Stresses due to casting of deck slab and cross-girders

Weight of deck slab $= 9.0 \times 1.0 \times 1.0 \times 0.20 \times 2400 = 4320$ Kg/m

Weight of cross-girder $= 2 \times 1.45 \times 2.2 \times 0.2 \times 2400 = 3060$ Kg/m

$$R_A = \frac{1}{2}(4320 \times 39.0 + 5 \times 3060) = 91{,}890 \text{ Kg}$$

$\therefore$ Moment at midspan $= 91{,}890 \times 19.5 - 4320 \times 19.5 \times \dfrac{19.5}{2} - 3060(13.0 + 6.5)$

$= 17{,}91{,}855 - 8{,}21{,}340 - 59{,}670 = 9{,}10{,}850$ Kg.m

Moment per girder assuming equal sharing $M_s = \dfrac{9{,}10{,}850}{3} = 3{,}03{,}600$ Kg.m

$$\therefore \quad \sigma_{tg} = \frac{M_S}{Z_t} = \frac{3{,}03{,}600 \times 100}{6{,}44{,}300} = (+)\ 47.12\ \text{Kg/cm}^2$$

$$\sigma_{bg} = \frac{M_S}{Z_b} = \frac{3{,}03{,}600 \times 100}{5{,}31{,}400} = (-)\ 57.13\ \text{Kg/cm}^2$$

Step 15 Stresses in girder with self-wt. of girder, weight of deck slab and cross-girder plus 1st stage prestress with partial loss plus 2nd stage prestress (Step 13 + Step 14)

$\sigma_{tg} = (+)\ 26.36 + 47.12 = (+)\ 73.48\ \text{Kg/cm}^2 \qquad \sigma_{bg} = (+)\ 109.57 - 57.13 = (+)\ 56.14\ \text{Kg/cm}^2$

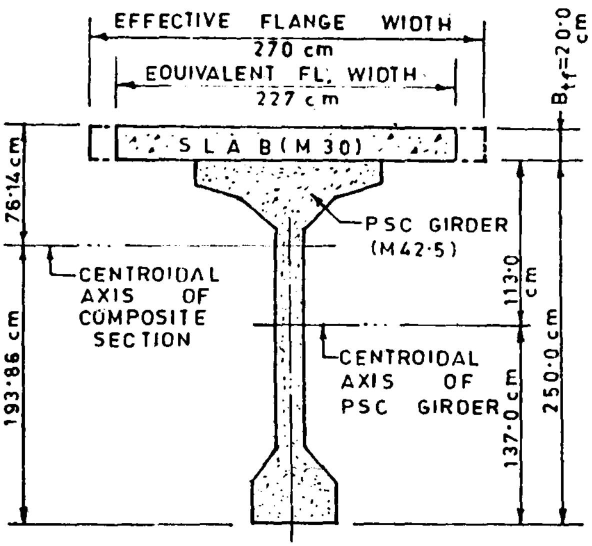


FIG.16·18 DETAILS OF COMPOSITE GIRDER
(Example 16·1)

Permissible max. and min. temporary stresses are (+) 195 Kg/cm^2 and (–) 19.5 Kg/cm^2 (Refer Step 13)

Step 16 Sectional properties of the Composite Section

Actual effective flange width as per Clause 604.1 of IRC : 22-1986 (Section VI – Composite Construction) and Clause 305.12.2 of IRC : 21-1987 (Section III – Cement Concrete) is minimum of the following :

i) $\frac{1}{4}$ of effective span $= \frac{1}{4} \times 39.0 = 9.75$ m = 975 cm.

ii) Distance between center of ribs = 2.7 m = 270 cm.

iii) The breadth of rib + 12 times the thickness of slab = 19.0 + 12 × 30 (average value including flange of PSC girder) = 379 cm.

Hence, 270 cm. is the least value. The equivalent flange width as per Art. 16.4.3 is given by :

Equivalent flange width = Eff. flange width $\times \frac{Ec(s)}{Ec(p)} = 270 \times \frac{5700\sqrt{30}}{5700\sqrt{42.5}} = 227$ cm.

TABLE 16.11 : AREA OF CROSS-SECTION, y_t AND y_b OF COMPOSITE SECTION (FIG. 16.18)

Item	Area (cm^2)	$\bar{x}$ = CG of girder/slab from bottom (cm)	$a\bar{x}$(cm)^3	y_b = CG of composite section from bottom (cm)
Precast girder (Refer Table 16.8)	8690	137	11,90,530	$y_b = \frac{\Sigma A\bar{x}}{\Sigma A} = \frac{23,70,930}{13,230} = 193.86$
In-situ slab	227 × 20 = 4540	260	11,80,400	$y_t = 270 - 193.86 = 76.14$
$\Sigma A = 13,230$		$\Sigma A\bar{x} = 23;70,930$		

TABLE 16.12 : MOMENT OF INERTIA AND SECTION MODULII OF COMPOSITE SECTION (FIG. 16.18)

Item	Io (own axis) (cm^4)	$\bar{x}$ = C.G. distance of each member from CG of composite section (cm)	$A\bar{x}^2$ (cm^4)	$I = I_o + A\bar{x}^2$ (cm^4)
Precast girder (Refer Table 16.9)	7,28,08,700	193.96 − 137 = 56.86	2,80,95,300	10,09,04,000
In-situ slab	$\frac{227 \times 20^3}{12}$ = 1,51,300	260 − 193.86 = 66.14	1,97,72,700	1,99,24;000
			ΣI = 12,08.28,000 = 1;20,828 × 10^3 cm^4	

Z_{ts} (top of slab) $= \frac{1,20,828 \times 10^3}{76.14} = 15,86,900 \text{ cm}^3$ $\quad Z_{tg}$ (top of girder) $= \frac{1,20,828 \times 10^3}{56.14} = 21,52,300 \text{ cm}^3$

$$Z_{bg} \text{ (bottom of girder)} = \frac{1,20,828 \times 10^3}{193.86} = 6,23,300 \text{ cm}^3$$

Step 17 Moment due to footway slab, kerb, precast footway slab, railing etc.

UDL = Weight per m = 2380 Kg (Calculations not shown)

$$\therefore \quad \text{Moment at midspan} = \frac{2380 \times (39)^2}{8} = 4,76,000 \text{ Kgm}$$

Moment per girder (assumed equal sharing) $= \frac{4,76,000}{3} = 1,58,667$ Kgm.

Step 18 Stresses due to footway slab, kerb etc.

$$6_{ts} = \frac{1,58,667 \times 100}{15,86,900} = (+)\ 10.00\ \text{Kg/cm}^2 \qquad 6_{tg} = \frac{1,58,667 \times 100}{21,52,300} = (+)\ 7.37\ \text{Kg/cm}^2$$

$$6_{bg} = \frac{1,58,667 \times 100}{6,23,300} = (-)\ 25.46\ \text{Kg/cm}^2$$

Step 19 Stress in Composite Section due to self wt. of girder + 1st stage prestress with partial loss + 2nd stage prestress + weight of deck slab, cross girder, footway slab, kerb etc. (Step 15 + Step 18).

$$6_{ts} = 0 + 10.00 = (+)\ 10.00\ \text{Kg/cm}^2 \qquad 6_{tg} = (+)\ 73.48 + 7.37 = (+)\ 80.85\ \text{Kg/cm}^2$$

$$6_{bg} = (+)\ 56.14 - 25.46 = (+)\ 30.68\ \text{Kg/cm}^2$$

Step 20 Partial Losses of 1st & 2nd stage prestress at 90 days after casting of girder before 3rd stage e prestress.

(a) ***Partial loss for 1st stage cable***

i) *Due to creep*

Creep strain per 10 MPa at 100% maturity (already considered previously)	4.0×10^{-4}
Creep strain per 10 MPa at 110% maturity at 90 days	3.6 × 10– 4
Difference	0.4×10^{-4}

Stress at bottom of girder before casting footway slab etc. = 56.14 Kg/cm² (Step 15)

$\therefore$ Stress at C.G. of cable $= 56.14 \times \frac{(137 - 7.5 - 7.14)}{137} = 50.14\ \text{Kg/cm}^2 = 4.91$ MPa

$\therefore$ Differential creep strain $= 0.4 \times 10^{-4} \times \frac{4.91}{10} = 0.20 \times 10^{-4}$

$\therefore$ Loss of stress = Es × Strain $= 2.1 \times 10^5 \times 0.20 \times 10^{-4} = 4.15$ MPa $= 0.42\ \text{Kg/mm}^2$

$\therefore$ Percentage loss of prestress $= \frac{0.42}{88.5} \times 100 = 0.48$

ii) *Loss due to Shrinkage*

Residual shrinkage at 28 days when 2nd stage prestressing is done	1.9 × 10– 4
Residual shrinkage at 90 days when 3rd stage prestressing is done	1.5×10^{-4}
Difference	0.4×10^{-4}

$\therefore$ Loss of stress = Es × strain $= 2.1 \times 10^5 \times 0.4 \times 10^{-4} = 8.4$ MPa $= 0.86$ Kg. per mm²

$\therefore$ Percentage loss $= \frac{0.86}{88.5} \times 100 = 0.97$

iii) *Loss due to Relaxation of Steel*

Already considered previously and no further loss in 1st stage prestress on this account will occur.

Total loss for (i) to (iii) = 0.48 + 0.97 = 1.45%

(b) ***Partial loss for 2nd Stage Cable***

Loss due to creep and shrinkage same as 1st stage cable, i.e. 0.48 + 0.97 = 1.45%.

Loss due to relaxation of steel

Relaxation loss with prestress of 0.56 UTS = 21 MPa = 2.14 Kg/mm^2

$\therefore$ Percentage loss of prestress $= \dfrac{2.14}{88.5} \times 100 = 2.42$

$\therefore$ Total partial loss = 1.45 + 2.42 = 3.87%

Step 21 Stresses in Composite Section with second partial loss of 1st stage cable and first partial loss of 2nd stage cable

These partial losses will occur when the composite deck is in action and therefore, the loss of prestress may be considered as a tensile force, i.e., opposite to the prestressing force acting at the respective CG of the cables on the composite section. These tensile forces will cause a direct tension and bending stress due to sagging moment.

From Table 16.10, total prestressing force for 1st and 2nd stage cables = 286.02 × 10^3 Kg.

$\therefore$ Tensile force for 1st stage cables = 286.02 × 10^3 × percentage loss = $286.02 \times 10^3 \times \dfrac{1.45}{100}$ = 4150 Kg acting at 14.64 cm from bottom.

$\therefore$ Moment due to the tensile force on the composite section = 4150x (193.86 – 14.64) = 7,43,800 Kgcm.

Tensile force for 2nd Stage cables = $286.02 \times 10^3 \times \dfrac{3.87}{100}$ = 11,070 Kg acting at 18.21 cm from bottom.

Moment due to this force = 11,070 (193.86 – 18.21) = 19,44,400 Kg.cm.

Total tensile force for 1st & 2nd Stage cables = 4150 + 11,070 = 15,220 Kg.

Total moment = 7,43,800 + 19,44,400 = 26,88,200 Kg.cm.

$$\therefore \quad 6_u = (-)\frac{15{,}220}{13{,}230} + \frac{26{,}88{,}200}{15{,}86{,}900} = (-)\ 1.15 + 1.69 = (+)\ 0.54 \text{ Kg/cm}^2$$

$$6_{tg} = (-)\frac{15{,}220}{13{,}230} + \frac{26{,}88{,}200}{21{,}52{,}300} = (-)\ 1.15 + 1.25 = (+)\ 0.10 \text{ Kg/cm}^2$$

$$6_{bg} = (-)\frac{15{,}220}{13{,}230} - \frac{26{,}88{,}200}{6{,}23{,}300} = (-)\ 1.15 - 4.31 = (-)\ 5.46 \text{ Kg/cm}^2$$

Step 22 Stresses in Composite Section with prestress and loading as in steps 19 and 21, i.e., with partial loss of 1st and 2nd stage prestress before 3rd stage prestress.

6_u = + 10.00 + 0.54 = (+) 10.54 Kg/cm^2 $\quad$ 6_{tg} = + 80.85 + 0.10 = (+) 80.95 Kg/cm^2

6_{bg} = + 30.68 – 5.46 = (+) 25.22 Kg/cm^2

Step 23 3rd Stage Prestress (Ref. Fig. 16.16 & 16.18 and Table 16.11 & 16.12)

Nos. of cables in 3rd stage are 7 (Ref. Step 6) and stress in 3rd Stage cable is 43.7 × 10^3 Kg. vide Step 7.

$\therefore$ Prestress force $= 7 \times 43.78 \times 10^3 = 306.46 \times 10^3$ Kg.

C.G. of 7 cables from C.G. of composite section (Fig. 16.16 and 16.18)

$$= 193.86 - \frac{(1 \times 22.5 + 3 \times 27.5 + 3 \times 37.5)}{7} = 193.86 - 31.07 = 162.79 \text{ cm}$$

$\therefore$ Stresses due to 3rd Stage prestress are :

$$\sigma_{ts} = \frac{306.46 \times 10^3}{13{,}230} - \frac{49{,}890 \times 10^3}{15{,}86{,}900} = (+)\ 23.16 - 31.44 = (-)\ 8.28 \text{ Kg/cm}^2$$

$$\sigma_{tg} = \frac{306.46 \times 10^3}{13{,}230} - \frac{49{,}890 \times 10^3}{21{,}52{,}300} = (+)\ 23.16 - 23.18 = (-)\ 0.02 \text{ Kg/cm}^2$$

$$\sigma_{bg} = \frac{306.46 \times 10^3}{13{,}230} + \frac{49{,}890 \times 10^3}{6{,}23{,}300} = (+)\ 23.16 + 80.19 = (+)\ 103.35 \text{ Kg/cm}^2$$

Step 24 Stresses in the Composite Section after 3rd stage prestress (Step 22 + Step 23)

$$\sigma_{ts} = (+)\ 10.54 - 8.28 = (+)\ 2.26 \text{ Kg/cm}^2 \qquad \sigma_{tg} = (+)\ 80.95 - 0.02 = (+)\ 80.93 \text{ Kg/cm}^2$$

$$\sigma_{bg} = (+)\ 25.22 + 103.35 = (+)\ 128.57 \text{ Kg/cm}^2$$

Permissible temporary stresses in concrete are (+) 195.0 Kg/cm^2 (compression) and (-) 19.50 Kg/cm^2 (tension) for M 42.5 concrete (i.e., precast girder). For deck concrete of M30 grade, these values may be taken as + 135.0 Kg/cm^2 (compression) and (–) 13.5 Kg/cm^2 (tension). Hence, the stresses are within permissible limits.

Step 25 Residual losses of 1st, 2nd and 3rd stages of prestressing

(a) ***Loss due to creep***

(i) *1st Stage Cables*

Balance creep strain $= 3.6 \times 10^{-4}$ per 10 MPa. From Step 24, concrete stress at bottom of girder = (+) 116.45 Kg/cm^2

$\therefore$ Stress at the C.G. of 1st Stage Cable

$$= (+)\ 114.15 \times \frac{(193.86 - 14.64)}{193.86} = (+)\ 107.66 \text{ Kg/cm}^2$$

This stress will gradually reduce due to loss. Let us take, a value of (+) 100.00 Kg/cm^2 i.e., 9.8 MPa for the calculation of loss.

$$\therefore \quad \text{Creep strain} = 3.6 \times 10^{-4} \times \frac{9.8}{10} = 3.53 \times 10^{-4}$$

$$\therefore \quad \text{Loss of stress} = E_S \times \text{strain} = 2.1 \times 10^5 \times 3.53 \times 10^{-4} = 74.13 \text{ MPa} = 7.56 \text{ Kg/mm}^2$$

$$\therefore \quad \text{Percentage loss} = \frac{7.56}{88.5} \times 100 = 8.5$$

ii) *2nd Stage Cables*

Balance creep $= 3.6 \times 10^{-4}$ per 10 MPa

$$\text{Stress at C.G. of 2nd Stage Cables} = (+)\ 114.15 \times \frac{(193.86 - 18.21)}{193.86} = (+)\ 105.56 \text{ Kg/cm}^2$$

The effective stress after loss may be taken same as in 1st Stage cables. Hence, percentage of loss for 2nd Stage cables is the same as for 1st Stage cables, i.e. 8.5.

iii) *3rd Stage Cables*

Balance creep after 90 days of casting of girder when 3rd Stage cables are stressed is also 3.6×10^{-4} per 10 MPa. Since the concrete stress at C.G. of 3rd Stage cables will be less, a percentage of loss of 8.0 may be taken for 3rd Stage cables.

(b) ***Loss due to Shrinkage Strain***

Balance shrinkage strain for 1st and 2nd Stage cables and shrinkage strain for 3rd Stage cables after 90 days is the same viz. 1.5×10^{-4} (Table 16.3).

Hence loss of stress = Es × strain = $2.1 \times 10^5 \times 1.5 \times 10^{-4} = 31.5$ MPa = 3.21 Kg/cm^2

∴ Percentage loss for 1st and 2nd Stage cables = $\dfrac{3.21}{88.5} \times 100 = 3.6$

Percentage loss for 3rd Stage cables = $\dfrac{3.21}{94.8} \times 100 = 3.4$

(c) ***Loss due to relaxation of steel***

This loss for 1st and 2nd Stage cables has already occurred and hence no further consideration in this regard is necessary. Relaxation loss for 3rd Stage cables for initial prestress of 0.6 of UTS is 35 MPa (Ref. Table 16.4).

∴ Percentage loss = $\dfrac{35}{9.8 \times 94.8} \times 100 = 3.8$

Summary of final losses :

1st Stage cables = 8.5 + 3.6 + 0 = 12.1%

2nd Stage cables = 8.5 + 3.6 + 0 = 12.1%

3rd Stage cables = 8.0 + 3.4 + 3.8 = 15%

Step 26 Stresses in Composite Section due to residual losses of prestress of all stages

These stresses will be determined as in Step 21. Tensile force for 1st Stage cables, = $286.02 \times 10^3 \times \dfrac{12.1}{100}$ = 34,600 Kg acting at 14.64 cm. from bottom. Moment due to this force = 34,600 × (193.86 − 14.64) = 6201 × 10^3 kg.cm...

Tensile force for 2nd stage cable same as for 1st Stage cable i.e. 34,600 Kg. acting at 18.21 cm. from bottom.

∴ Moment due to this = 34,600 × (193.86 − 18.21) = 6077 × 10^3 Kg.cm. Tensile force for 3rd stage cables = $306.46 \times 10^3 \times \dfrac{15.2}{100}$ = 46,580 Kg. acting at 31.07 cm from bottom.

∴ Moment due to this = 46,580 × (193.86 − 31.07) = 7583 × 10^3 Kg.cm.

Total tensile force = 34,600 + 34,600 + 46,580 = 1,15,780 Kg.

Total moment (sagging) = (6201 + 6077 + 7583) × 10^3 = 19,861 × 10^3 Kg.cm.

Hence, the stresses in the composite section due to residual losses of the cables of all stages are :

$$\sigma_{ts} = (-)\frac{1,15,780}{13,230} + \frac{19,861 \times 10^3}{15,86,900} = (-)\ 8.75 + 12.52 = (+)\ 3.77 \text{ Kg/cm}^2$$

$$\sigma_{tg} = (-)\frac{1,15,780}{13,230} + \frac{19,861 \times 10^3}{21,52,300} = (-)\ 8.75 + 9.23 = (+)\ 0.48 \text{ Kg/cm}^2$$

$$\sigma_{bg} = (-)\frac{1,15,780}{13,230} - \frac{19,861 \times 10^3}{6,23,300} = (-)\ 8.75 - 31.86 = (-)\ 40.61 \text{ Kg/cm}^2$$

Step 27 Stresses in Composite Section with prestress and loading as in Step 24 with residual losses for cables of all stages as in Step 25.

$$\sigma_{ts} = (+)\ 2.26 + 3.77 = +\ 6.03 \text{ Kg/cm}^2 \qquad \sigma_{tg} = (+)\ 80.93 + 0.48 = +\ 81.41 \text{ Kg/cm}^2$$

$$\sigma_{bg} = (+)\ 128.57 - 40.61 = +\ 87.96 \text{ Kg/cm}^2$$

Step 28 Stresses in Composite Section due to wearing course on bridge deck.

Weight of wearing course (average thickness 85 mm) per m. of deck = 7.5 × 0.085 × 1.0 × 2400 = 1530 Kg

$$\therefore \quad \text{Moment at mid span} = \frac{1530 \times (39.0)^2}{8} = 2,90,900 \text{ Kgm.}$$

$$\therefore \quad \text{Moment per girder} = \frac{2,90,900}{3} = 96,970 \text{ Kgm.}$$

Hence, stresses are :

$$\sigma_{ts} = \frac{96,970 \times 100}{15,86,900} = (+)\ 6.11 \text{ Kg./cm}^2 \qquad \sigma_{tg} = \frac{96,970 \times 100}{21,52,300} = (+)\ 4.51 \text{ Kg/cm}^2$$

$$\sigma_{bg} = \frac{96,970 \times 100}{6,23,300} = (-)\ 15.56 \text{ Kg/cm}^2$$

Step 29 Stresses in the Composite Section at Service without Live Loads (Step 27 + Step 28)

$$\sigma_{ts} = 6.03 + 6.11 = (+)\ 12.14 \text{ Kg/cm}^2 \qquad \sigma_{tg} = 81.41 + 4.51 = (+)\ 85.92 \text{ Kg/cm}^2$$

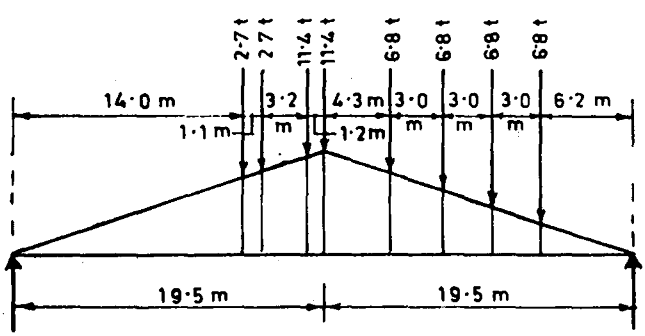


FIG. 16·19 CLASS A LOAD ON B.M. INFLUENCE LINE DIAGRAM AT MIDSPAN (Example 16·1)

$$6_{bg} = (+)\ 87.96 - 15.56 = (+)\ 72.4\ \text{Kg/cm}^2$$

Step 30 Moments due to Live Loads

(a) ***2-Lanes of Class A Loading (Fig. 16.19)***

Live Load moment at mid span for 2 lanes loading

$$= 2 \times \frac{9.75}{19.5}\,[2.7\,(14.0 + 15.1) + 11.4\,(12.8 + 14.0) + 6.8\,(6.2 + 9.2 + 12.2 + 15.2)] = 675.13\ \text{tm}$$

$$\text{Impact factor} = \frac{4.5}{6 + L} = \frac{4.5}{6 + 39.0} = 0.10$$

$\therefore$ L.L. moment with impact $= 1.1 \times 675.13 = 742.64$ tm

Distribution factor for central girder as given = 1.10

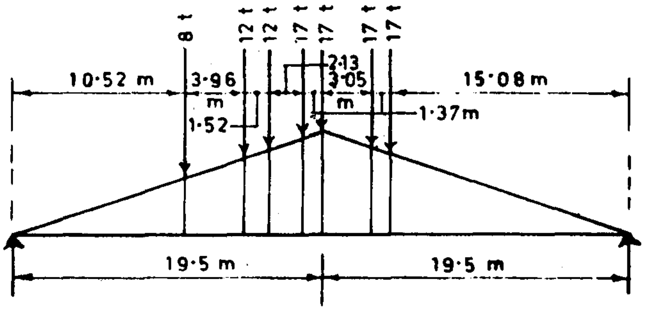


FIG. 16·20 CLASS 70R LOAD ON B.M. INFLUENCE LINE DIAGRAM AT MIDSPAN (Ex. 16·1)

$$\therefore \quad \text{L.L. moment of central girder} = \frac{742.64}{3} \times 1.10 = 272.30\ \text{tm}$$

(b) ***Single lane of Class 70R wheeled vehicle (Fig. 16.20)***

Live load moment at midspan

$$= \frac{9.75}{19.5}\,[8 \times 10.52 + 12\,(14.48 + 16.0) + 17\,(18.13 + 19.5 + 15.08 + 16.45)] = 812.82\ \text{tm}$$

L.L. moment with impact $= 1.1 \times 812.82 = 894.10$ tm

Distribution factor for central girder as given = 1.10

$$\therefore \quad \text{L.L. moment on central girder} = \frac{894.10}{3} \times 1.10 = 327.84\ \text{tm}$$

Step 31 Moments due to Footway Loading

As per Art. 5.5.3 Chapter 5, Intensity of load, $P = P' - 260 + \frac{4800}{L}\left(\frac{16.5 - W}{15}\right)$

$$= \left(400 - 260 + \frac{4800}{39}\right)\left(\frac{16.5 - 7.5}{15}\right) = 263 \times 0.6 = 158\ \text{Kg/m}^2$$

Total footway loading for two footpaths of 1.5 m = 2 × 1.5 × 158 = 474 Kg/m

$$\therefore \quad \text{Moment due to footway loading} = \frac{474 \times (39.0)^2}{8} = 90{,}100 \text{ Kg.m.} = 90.1 \text{ tm}$$

$$\therefore \quad \text{Moment per girder} = \frac{90.1}{3} = 30.1 \text{ tm}$$

Step 32 Design Live Load Moment and Stresses due to L.L. moment in the Composite Section

Single lane of Class 70R loading produces more moment than two lanes of Class A loading (Ref. Step 30 and the max. live load moment is 327.84 tm).

∴ Design moment = Moment due to live load + moment due to footpath loading.

= 327.84 + 30.1 = 417.94 tm = 417.94 × 10^5 Kg.cm.

$$\therefore \quad 6_{u} = \frac{417.94 \times 10^5}{15{,}86{,}900} = (+)\ 26.33 \text{ Kg/cm}^2 \qquad 6_{tg} = \frac{417.94 \times 10^5}{21{,}52{,}300} = (+)\ 19.42 \text{ Kg./cm}^2$$

$$6_{bg} = \frac{417.94 \times 10^5}{6{,}23{,}300} = (-)\ 67.06 \text{ Kg/cm}^2$$

Step 33 Stresses in the Composite Section at Service with Live Load (Step 29 + Step 32)

$$6_{u} = (+)\ 12.14 + 26.33 = (+)\ 38.47 \text{ Kg./cm}^2 \qquad 6_{tg} = (+)\ 85.92 + 19.42 = (+)\ 105.34 \text{ Kg/cm}^2$$

$$6_{bg} = (+)\ 72.40 - 67.06 = (+)\ 5.34 \text{ Kg/cm}^2$$

Step 34 Permissible and Actual Stresses at Service

As per Art. 5.21.2.3, compressive stress during service shall not exceed 0.33 fck and no tensile stress shall be permitted. Therefore, max. compressive permissible stresses are 14.03 MPa (or 143.16 Kg/cm^2) for M42.5 grade concrete (PSC girder) and 10 MPa (or 102.04 Kg/cm^2) for M30 grade concrete (in-situ deck slab). On examination of Step 29 and Step 33 it may be seen that the max. compressive stresses at service are 10.32 MPa (105.34 Kg/cm^2) for PSC girder and 3.77 MPa (38.47 Kg/cm^2) for in-situ slab. The minimum stresses are 0.52 MPa (or 5.34 Kg/cm^2) (compression) for PSC girder and 1.19 MPa (or 12.14 Kg/cm^2) (compression) for in-situ slab.

Step 35 Stresses in the PSC girder and the Composite Section at various stages of loading and prestressing.

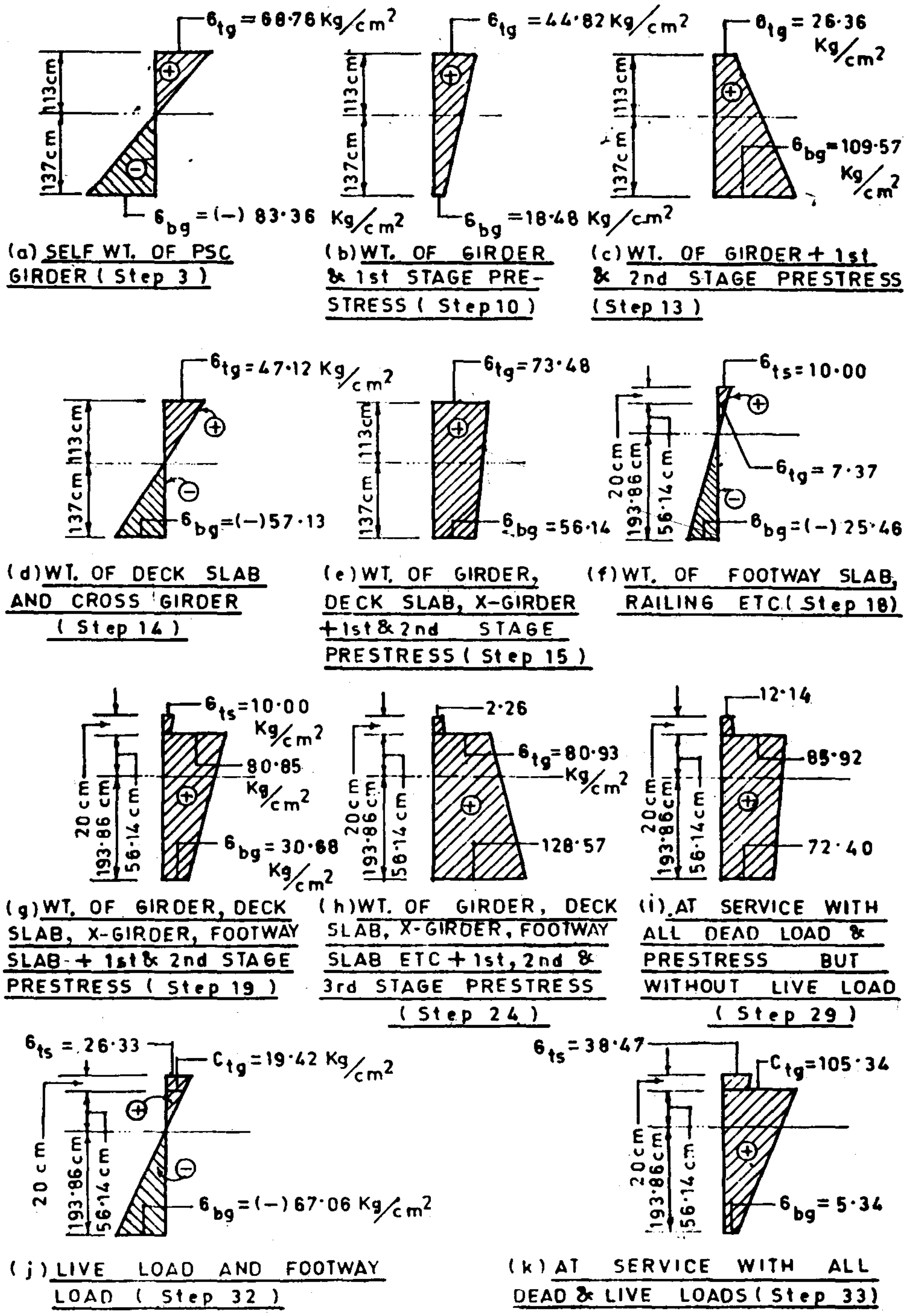
For better understanding of the development of stresses in the PSC girder and the composite section, diagramatic representation of the stresses is shown in Fig. 16.21.

Step 36 Ultimate strength of Composite Section

As per Art. 16.8.6.2, Ultimate Load = 1.25 G + 2.0 SG + 2.5 Q

∴ Moment due to G :

i)	Self wt. of PSC girder	4,43,000 Kgm (Step 02)
ii)	Wt. of slab and cross-girder	3,03,600 Kgm. (Step 14)
iii)	Wt. of footway Slab etc.	1,58,667 Kgm. (Step 17)
	Total :	9,05,267 Kgm.



(a) SELF WT. OF PSC GIRDER (Step 3)

(b) WT. OF GIRDER & 1st STAGE PRESTRESS (Step 10)

(c) WT. OF GIRDER + 1st & 2nd STAGE PRESTRESS (Step 13)

(d) WT. OF DECK SLAB AND CROSS GIRDER (Step 14)

(e) WT. OF GIRDER, DECK SLAB, X-GIRDER + 1st & 2nd STAGE PRESTRESS (Step 15)

(f) WT. OF FOOTWAY SLAB, RAILING ETC (Step 18)

(g) WT. OF GIRDER, DECK SLAB, X-GIRDER, FOOTWAY SLAB + 1st & 2nd STAGE PRESTRESS (Step 19)

(h) WT. OF GIRDER, DECK SLAB, X-GIRDER, FOOTWAY SLAB ETC + 1st, 2nd & 3rd STAGE PRESTRESS (Step 24)

(i) AT SERVICE WITH ALL DEAD LOAD & PRESTRESS BUT WITHOUT LIVE LOAD (Step 29)

(j) LIVE LOAD AND FOOTWAY LOAD (Step 32)

(k) AT SERVICE WITH ALL DEAD & LIVE LOADS (Step 33)

FIG. 16·21 DIAGRAMATIC REPRESENTATION OF STRESSES IN THE PSC GIRDER AND COMPOSITE SECTION (Ex. 16·1)

Moment due to SG :

Due to wearing course = 96,970 Kgm (Step 28)

Moment due to Q = 4,17,940 Kgm (Step 32)

$\therefore$ Ultimate moment = $1.25 \times 9{,}05{,}267 + 2.0 \times 96{,}970 + 2.5 \times 4{,}17{,}940 = 23{,}70{,}350$ Kgm.

Ultimate moment of resistance of concrete (vide Art. 16.8.6.2),

$$M_u = 0.17\ bd^2 f_{ck} + \frac{2}{3} \times 0.8\ (B_f - b) \times (d - \frac{t}{2}) \times t\ f_{ck}$$

In the above expression, the first term is for web portion and the second term is for flange portion. The second term will be used for the evaluation for Mu contributed by the flange of the T-beam of PSC section and the effective width of deck slab acting as flange of the composite section.

Mu for web = $0.176 \times 19.0 \times (228)^2 \times 425 = 73{,}900 \times 10^3$ Kg. cm.

Mu for flange of PSC beam (average thickness, 17.0 cm)

$$= \frac{2}{3} \times 0.8\ (125 - 19.0) \times (228 - \frac{17.0}{2}) \times 17.0 \times 425 = 89{,}700 \times 10^3 \text{ Kg.cm.}$$

Mu for effective width of deck slab = $\frac{2}{3} \times 0.8\ (270 - 19.0) \times (248 - 10) \times 20 \times 300 = 1{,}91{,}200 \times 10^3$ Kg.cm.

Total Mu for concrete = $73{,}900 + 89{,}700 + 1{,}91{,}200) \times 10^3$ Kg.cm.

$= 3{,}54{,}800 \times 10^3$ Kg.cm. = 35,48,000 Kg.m.

This is 50 percent more than the ultimate moment of 23,70,300 Kgm. Hence satisfactory.

Mu for steel = $0.9 \times d \times As \times fb$

$$= 0.9 \times 248\ (21 \times 12 \times \frac{\pi}{4} \times \frac{7.0^2}{100}) \times 158 \times 10^2$$

$\doteq 34{,}20{,}100 \times 10^2$ Kg.cm = 34,20,100 Kgm.

This is 44 percent more than the ultimate moment of 23,70,300 Kgm. but less then Mu of concrete equal to 35,48,000 Kgm. as required by Art. 16.8.6.2. Hence satisfactory.

Step 37 Nominal Non-tensioned Reinforcement

As per Art. 16.9.1, minimum reinforcement in the vertical direction shall not be less than 0.3 percent for mild steel or 0.18 percent for HYSD bars of the cross sectional area in plan.

As in web per m = $100 \times 19 \times \frac{0.18}{100} = 3.42$ cm^2 = 342 mm^2 HYSD bars

As in web in each face = $\frac{1}{2} \times 342 = 171$ mm^2

Use 8 Φ @ 200 (minimum spacing as required in Art. 16.9.3) which gives As = 250 mm^2.

As in bottom bulb per m = $100 \times 60 \times \frac{0.18}{100} = 10.8$ cm^2 = 1080 mm^2

As in each face = 540 mm^2

Use 10 Φ @ 200 in between 8 Φ @ 200 giving As = 640 mm^2.

As per Art. 16.9.2, minimum longitudinal reinforcement shall not be less than 0.15 percent for HYSD bars for grade of concrete less than M45.

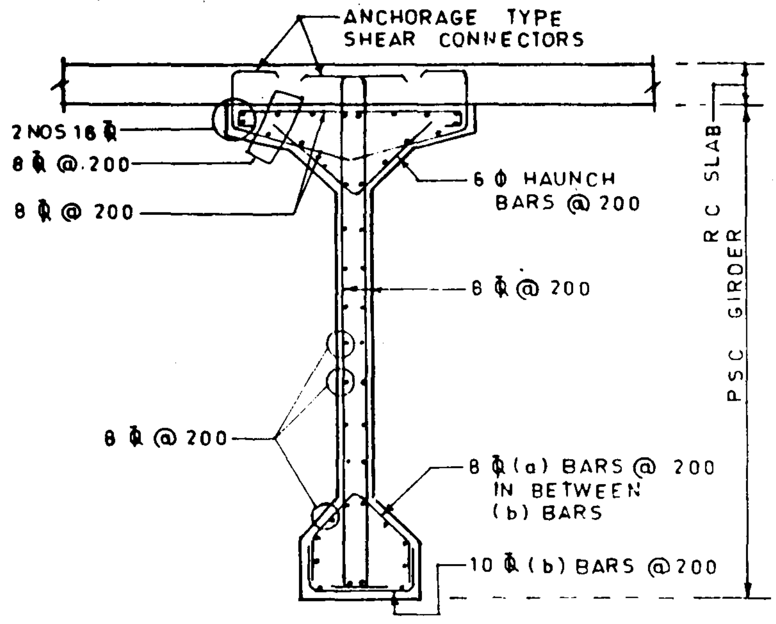


FIG.16·22 NOMINAL NON-TENSIONED REINFORCEMENT IN PRESTRESSED CONCRETE GIRDER (Ex.16·1)

$$\therefore \text{As per metre height} = 100 \times 19 \times \frac{0.15}{100} = 2.85 \text{ cm}^2 = 285 \text{ mm}^2$$

As in each face = 143 mm^2, use 8 bars at 200 (minimum spacing as per Art. 16.9.3) giving As = 250 mm^2. The nominal reinforcement details are shown in Fig. 16.22.

16.12 CABLE PROFILE

16.12.1 IRC : 18-1985 permits anchorage in deck surface. These anchorages are known as intermediate anchorages. However, IRC : SP-33 recommends that the stages of prestressing shall preferably be not more than two and no intermediate anchorages are permitted in the deck surface. Illustrative Example 16.1 is based on IRC : 18-1985 and has intermediate cable anchorages in the third stage. The cable profile shown in Fig. 16.23 is drawn accordingly details of which are presented in para 16.12.2 to 16.12.4.

16.12.2 For a simply supported girder, the moment at center is maximum and gets reduced to zero at support. Therefore, the prestressing cables placed at bottom with maximum eccentricity at the midspan are to be taken upwards with reduced eccentricity so that the resisting moment caused by the prestressing cable is reduced in relation with the actual moment in the beam. Generally, two-third of the cables are anchored at the ends of the girder and the remaining one-third is anchored into the deck. The former two-third cables are generally stressed before placing the girder in position and the latter one-third are stressed after casting and maturity of the deck slab. Approx. cable profile of PSC girder of the Illustrative Example 16.1 is shown in Fig. 16.23. Generally,

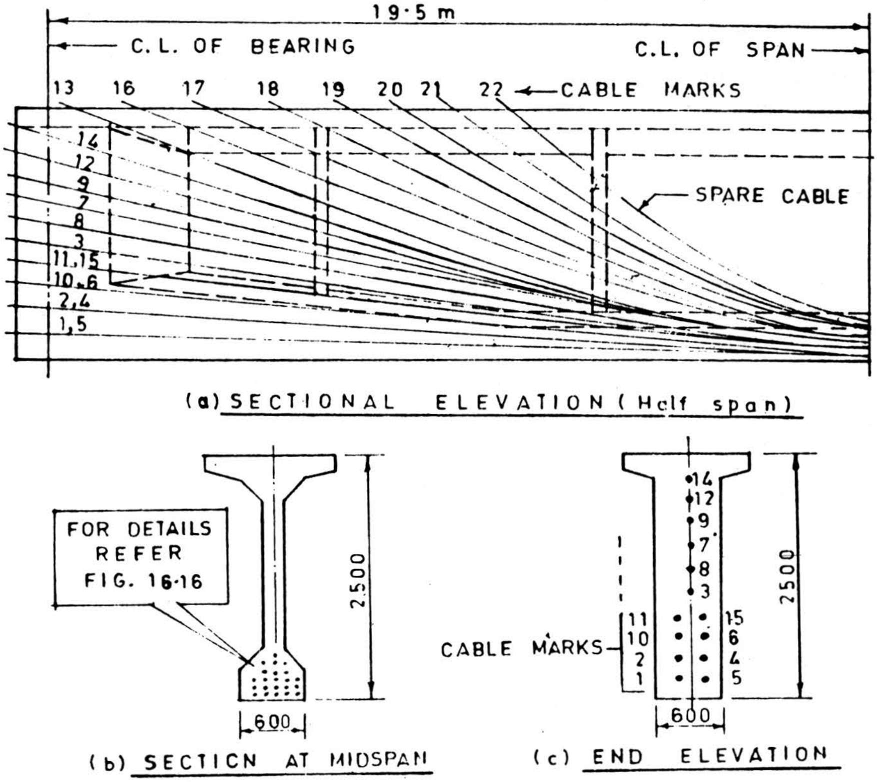
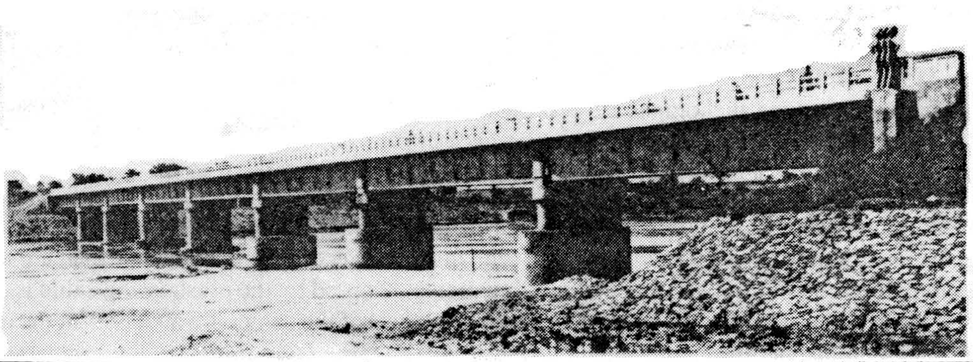


FIG.16·23 CABLE PROFILE—GENERAL ARRANGEMENT (Ex.16·1)

the cable profile is parabolic for simply supported girder as the moment diagram is also parabolic. A combination of straight and curved cable profile is also used.

16.12.3 In addition to the vertical curvature, the cables are required to be swayed horizontally by providing curvature in the horizontal plane in order to bring the cables toward the center of the girder for anchorage at the ends at or near the central axis of the girder. When the anchorage of the cable is to be done in pairs as in Fig. 16.23c, the depth of the bottom flange near the ends is to be increased to accommodate these twin cables near the ends as shown in broken line in Fig. 16.23a.



Photograph 4
BRIDGE OVER KANGSABATI RIVER
(Constructed on Krishnapur-Raipur Road in the district of Bankura, West Bengal. A prestressed Concrete Bridge having 6 (six) intermediate spans of 41.15 m and 2 (two) end spans of 40.61 m with total length of 328.12 m). (Highway Bridges)[2]

16.12.4 The spare cable, if not required to be stressed for additional prestress from design requirements (in case of short-fall of the main prestressing force), is removed and the duct is grouted.

16.13 PHOTOGRAPH OF A BRIDGE

16.13.1 Photograph 4 illustrates a T-beam prestressed concrete bridge having eight spans of 40 m (average).

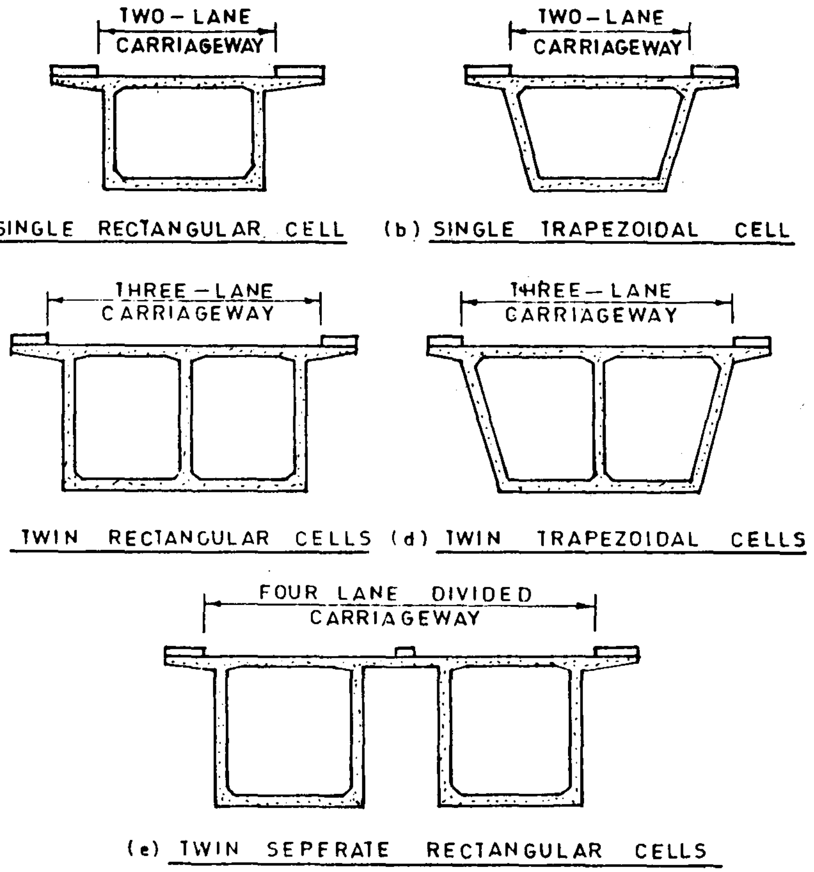


FIG.16·24 TYPES OF BOX-GIRDER DECKS

16.14 PRESTRESSED CONCRETE BOX-GIRDER BRIDGES

16.14.1 For larger spans, prestressed concrete box-girders are used instead of T-beams. These box-girders are normally constructed by *"Cantilever construction"* method. The girders are either prefabricated in sections and erected at site or cast in-situ in sections. The sections are erected or cast symmetrically from the pier for stability of the superstructure, pier and the foundation and *"stitched"* to the previous section by means of prestressing cables. Types of box-girders normally used are shown in Fig. 16.24. The box-girder shown in Fig. 16.24a and 16.24b are for two lanes carriageway. The twin cell box girders shown in Fig. 16.24c and 16.24d

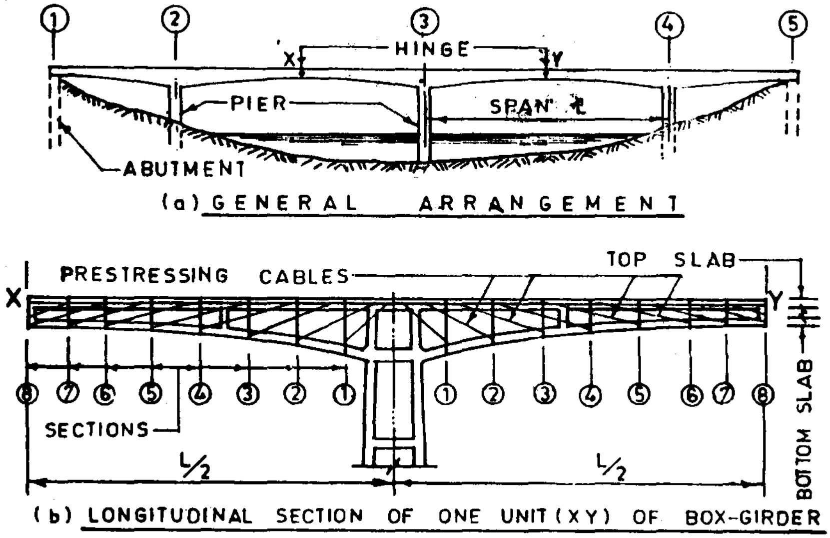


FIG.16·25 PSC BOX-GIRDER BRIDGE

may be adopted for six lanes divided carriageway when two such units are used side by side. The type shown in Fig. 16.24e may be used in four lanes divided carriageway.

16.14.2 Long section of a box-girder bridge constructed by the cantilever method is shown in Fig. 16.25a. Figures below the box-girder in Fig. 16.25b indicate units and sequence of construction from the piers. Arrangement of post-tensioned prestressing cables is also shown in Fig. 16.25b. Details of construction is dealt in Chapter 24.

16.15 REFERENCES

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CHAPTER 17

LONG SPAN BRIDGES

17.1 GENERAL

17.1.1 Table 1.5 of Chapter 1 shows that steel bridges have larger spans than the concrete bridges. For example, as may be seen in the same table, concrete bridges have a maximum spans of 78 metres for girder bridges, 230 metres for prestressed girder bridges and 305 metres for arch bridges whereas steel bridges have a maximum span of 140 metres for tubular girder bridges. 261 metres for continuous plate girder bridges, 376 metres for continuous truss bridges, 400 metres for steel girder cable-stayed bridges, 519 metres for arch bridges, 549 metres for cantilever bridges and 1298 metres for suspension bridges. Therefore, it is needless to say that long span bridges must be either all steel bridges or have steel as the main load bearing structural components.

17.1.2 Concrete bridges have the advantage that they require little maintenance cost compared to the steel bridges. That is why concrete bridges are built for small and medium span bridges but where long span bridges are required to be constructed for technical and other grounds, steel bridges or predominantly steel bridges are the only choice.

17.2 TYPES OF LONG SPAN BRIDGES

17.2.1 The long span bridges may be broadly classified in the following types :-

i) Continuous plate girder bridges.
ii) Continuous steel tubular or box-girder bridges.
iii) Steel arch bridges.
iv) Continuous or cantilever truss bridges.
v) Cable-stayed bridges.
vi) Suspension bridges.

17.3 CONTINUOUS PLATE GIRDER BRIDGES

17.3.1 Simply supported plate girder bridges have been discussed in Chapter 14. The basic design principles for continuous plate girder bridges remaining the same, the effect of stress reversal over supports due to continuity of the structure has to be duly considered in the design. Moreover, due to longer spans and continuity of the deck, large movement of the deck has to be duly catered for in the design of expansion joints and free bearings.

17.3.2 Salient features of one continuous plate girder bridge having 261 metres main span and 75 metres side spans are given below. This is the sava I bridge at Belgrade, Yugoslavia constructed in 1956 (Fig. 17.1).

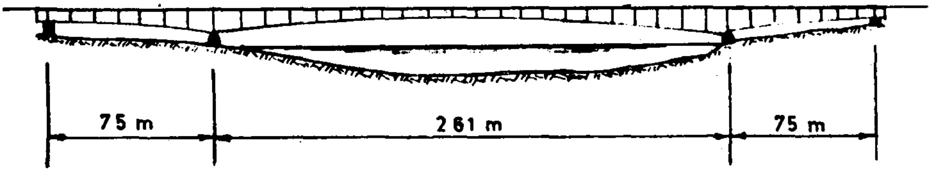


FIG. 17·1 SAVA I BRIDGE AT BELGRADE, YUGOSLAVIA (O' Connor)[6]

Sava I Bridge at Belgrade

The bridge has a carriageway of 12.0 metres with 3.0 metres footpaths on either side. Depths of girder are 4.72 metres at abutment, 4.57 metres at center of main span and 9.76 metres at pier. The span-depth ratio of the girder varies from 57 to 27. The bridge deck is orthotropic steel deck consisting of plate 10 mm to 18 mm thick stiffened by ribs at 305 mm centers. The thickness of the web plate is 14 mm. Vertical web stiffeners are placed at 9.0 metres center to center while the horizontal web stiffeners are at 760 mm centers approximately in the compression zone. List of some continuous plate girder bridges is shown in Table 17.1.

TABLE 17.1 : LIST OF SOME CONTINUOUS PLATE GIRDER BRIDGES

Sl. No.	Name of Bridge	Location	Type	Max. Span
1	Whiskey Creek	USA	Balanced cantilever	107 metres
2	Calcacieu	USA	Continuous plate girder	137 metres
3	Wiesladen-Schierstein	West Germany	Do	205 metres
4	Sava I	Yugoslavia	Do	261 metres

17.4 CONTINUOUS STEEL TUBULAR OR BOX-GIRDER BRIDGES

17.4.1 Tubular or box-girder bridges are so called for the shape of the girders which is tubular or box section. Various shapes of tubular or box-girder bridges are shown in Fig. 17.2.

Single rectangular box-section shown in Fig. 17.2a was adopted for the Europa Bridge over Sill Valley, Australia while double rectangular box-section (Fig. 17.2b) was adopted for the San Mateo-Hayward Bridge, USA. The single partitioned trapezoidal box-sections as shown in Fig. 17.2d and 17.2c were used for Concordia Bridge Montreal and Wuppertal Bridge, Germany, respectively.

17.4.2 The box-girders possess high torsional stiffness and strength compared to open cross-sections such as plate girders. The box-sections having a bottom plate connecting the bottom flanges require no scaffolding for the maintenance of the internal space as these are directly accessible from one end to the other. The girders of

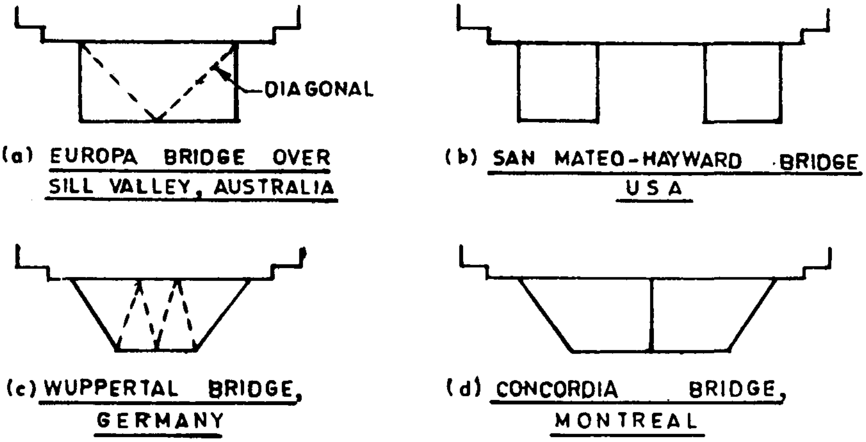


FIG. 17·2 VARIOUS BOX-GIRDER CROSS-SECTIONS

open cross-section have no such advantage and scaffolding is required for the maintenance of the internal space.

17.4.3 Brief particulars of one box-girder bridge viz. San Mateo-Hayward Bridge, USA are given below :

San Mateo-Hayward Bridge, USA

The bridge was constructed in 1967. The span arrangement and the cross-section of the bridge are shown in Fig. 17.3. The bridge has an orthotropic steel deck. The depth of the girder at center of main span is 4.57 metres and at pier is 9.15 metres thus giving the span-depth ratio from 50 to 25. List of some box-girder bridges are given in Table 17.2.

TABLE 17.2 : LIST OF SOME STEEL BOX-GIRDER BRIDGES

Sl. No.	Name of bridge	Location	Type	Max. span
1	Concordia Bridge	Montreal	Continuous	160 metres
2	Europa Bridge	Australia	Continuous	198 metres
3	San Mateo-Hayward Bridge	USA	Balanced cantilever	229 metres
4	Zoo Bridge	West Germany	Continuous	259 metres

17.5 STEEL ARCH BRIDGES

17.5.1 The development of high strength structural steel made it possible to construct arch bridges of larger spans similar to other steel bridges. Steel arch bridges are classified depending upon the arrangement of the deck or arrangement of the structural system as described in Chapter 13 for concrete arch bridges. Steel arch bridges may, however, have either solid ribs or trussed ribs while the concrete arch bridges will have only solid ribs.

17.5.2 The advantages of using steel arch bridges over girder bridges are similar to those of concrete arch bridges which are described in Chapter 13. The basic design principles for steel arch bridges are also the

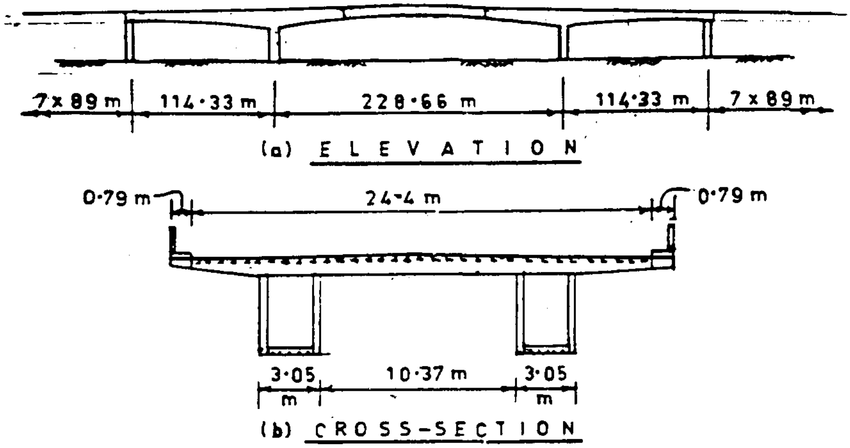


FIG.17·3 SAN MATEO-HAYWARD BRIDGE, USA (O'Connor)[6]

same as those for concrete arch bridges. However, the design considerations such as shrinkage of arch rib, creep etc. will not occur in steel arch bridges as in concrete bridges.

17.5.3 The salient features of two steel arch bridges are given below :

Rainbow Bridge

The bridge is located across Niagara River between Canada and United States of America, year of construction being 1941. The span and the rise of the bridge is shown in Fig. 17.4.

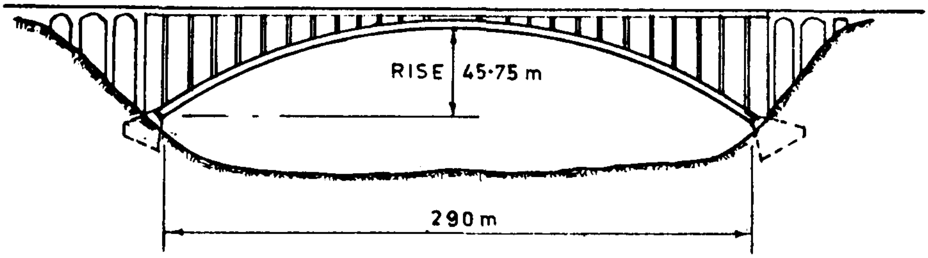


FIG. 17·4 RAINBOW BRIDGE (O'Connor)[6]

The arch is deck type with open spandrel having the arch rib fixed at the springing point. The arch rib consists of two number riveted steel box of 3.66 metres deep and 0.91 metres wide. These boxes are placed at a distance of 17.12 metres center to center. The bridge deck has a dual carriageway of 6.71 metres each separated by a median of 1.2 metres and a footpath of 3.0 metres on one side and safety kerb of 225 mm on the other side.

Port Mann Bridge

This bridge is situated near Vancouver, Canada, across Fraser River. The span arrangement of the bridge is shown in Fig. 17.5. The arch is a special type of tied arch having advantage of both the classic and tied arches. The arch is semi-through type thereby reducing the heights of both the suspenders and the spandrel columns. The carriageway of the bridge deck is 16.56 metres wide with 1.2 metres wide footpath on either side. List of some more arch bridges is given in Table 17.3.

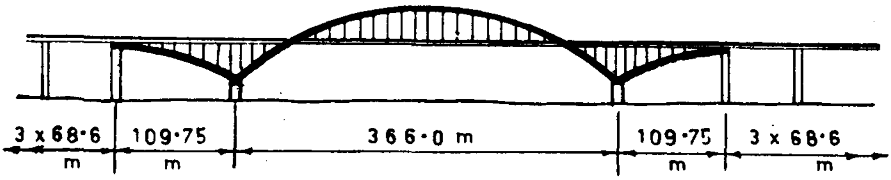


FIG.17·5 PORTMANN BRIDGE (O'Connor)[6]

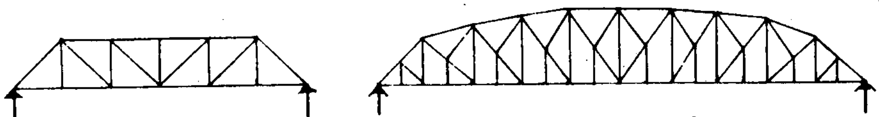
TABLE 17.3 : LIST OF SOME MORE STEEL ARCH BRIDGES

Sl. No.	Name of Bridge	Location	Type	Rise (m)	Span (m)
1	Kaiserlie Bridge	West Germany	Tied arch-tubular arch rib.	26.00	220
2	Askerofjord Bridge	Sweden	Fixed arch-tubular arch rib	40.5	278
3	Runcorn-Widnes Bridge	England	Cantilever trussed arch rib	—	330

17.6 CONTINUOUS OR CANTILEVER TRUSS BRIDGES

17.6.1 Types of simply supported truss bridges are shown in Fig. 14.6 in Chapter 14. Those types are also used for continuous as well as cantilever truss bridges. The basic principles of evaluating forces in the truss members are the same as explained in Art. 14.5. However, due to the presence of more members as well as on account of continuity the work becomes elaborate and time consuming.

17.6.2 For larger spans when the panel lengths are more, they are subdivided to give adequate supports for the deck. The Warren truss shown in Fig. 14.6a when used for larger spans, may be modified by providing verticals as shown in Fig. 14.6b for the aforesaid purpose. The Pettit is a modification of N or Pratt truss with subdivision of the panels (Fig. 17.6). K-truss has been used in Howrah Bridge which is a cantilever bridge (Fig. 17.8).



(a) N OR PRATT TRUSS (b) PETTIT TRUSS

FIG.17·6 SUBDIVISION OF TRUSS PANELS

17.6.3 The salient features of two long span steel truss bridges, one of continuous type and the other of cantilever type are described below :

Bridge over Fulda River

This bridge was constructed over Fulda River, West Germany. The span arrangement is shown in Fig. 17.7. The bridge has Warren trusses continuous over 7 spans shown in Fig. 17.7. Orthotropic steel deck integral with the top chord has been provided in the bridge. The trusses have a uniform depth of 6.0 metres for all spans thus giving a span-depth ratio of 23.8 for larger span. The deck has a carriageway of 9.0 metres with 1.75 metres footpath on either side as shown in Fig. 17.7.

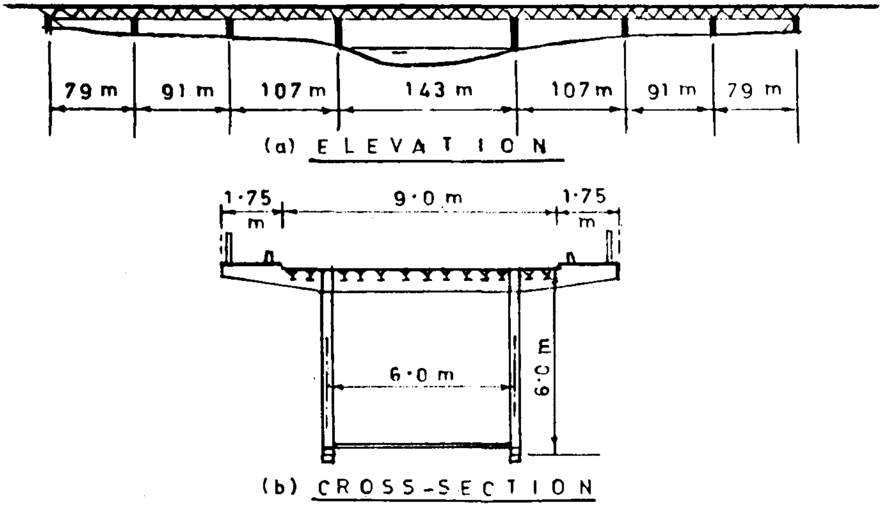


FIG. 17·7 BRIDGE OVER FULDA RIVER (O'Connor)[6]

Howrah Bridge

This bridge was constructed in 1943 over the Hooghly River at Calcutta. The span arrangement is shown in Fig. 17.8. The bridge has two end anchor spans (which are anchored at the end supports) and one main span consisting of two cantilevers and one suspended span. The bridge truss is a K-truss having panels subdivided for supporting the deck which is suspended by suspenders from panel joints. The deck is supported over longitudinal stringers resting on cross girders which are fixed to the suspenders. The cross-section of the deck is shown in Fig. 17.8b.

17.6.4 Table 17.4 shows some more continuous or cantilever steel truss bridges.

TABLE 17.4 : LIST OF SOME MORE CONTINUOUS OR CANTILEVER TRUSS BRIDGES *(Victor)*[10]

Sl. No.	Name of Bridge	Year of construction	Location	Type	Main span (metres)
1	Kingston Rhinecliff	1957	New York	Continuous truss	244
2	Brent Spence	1963	Ohio	Do	253
3	Tenmon	1966	Japan	Do	300
4	Astoria	1966	Oregan	Do	376
5	Transbay	1936	California	Cantilever truss	427
6	Greater New Orleans	1958	Louisiana	Do	480
7	Osaka Port	1974	Japan	Do	510

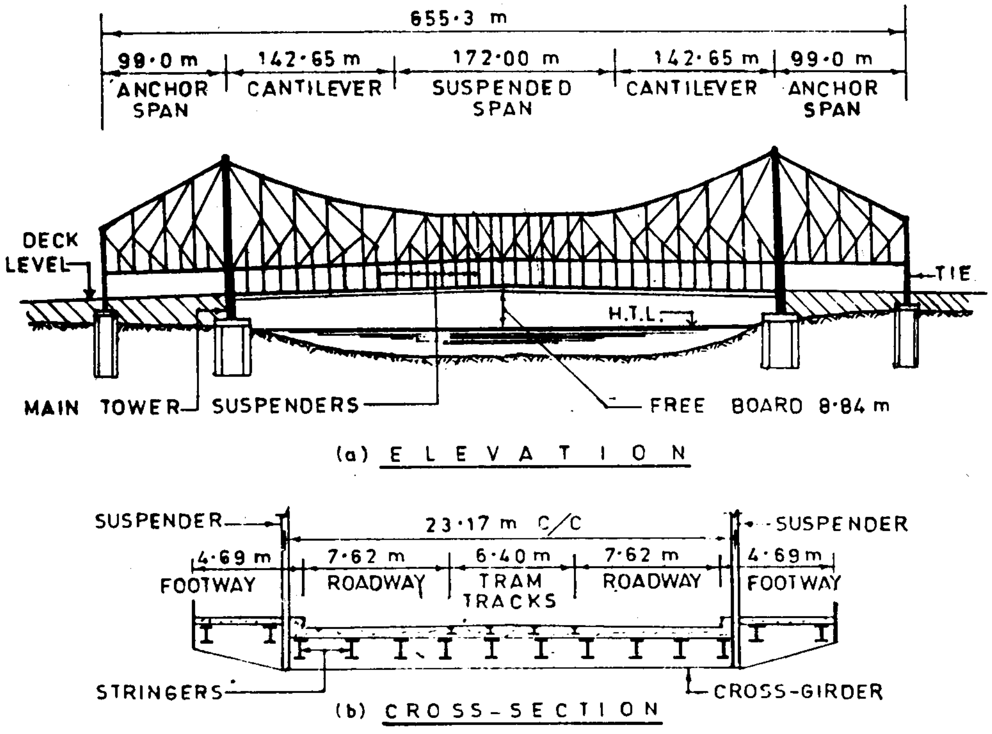


FIG.17·8 HOWRAH BRIDGE, CALCUTTA (Inst. of C.E.)[1]

17.7 CABLE STAYED BRIDGES

17.7.1 The cable-stayed bridges in the present form were constructed in Europe specially in West Germany after the Second World War when the need for reconstruction of a number of bridges was urgently felt. The cable-stayed bridges are suitable for the span range of 200 to 500 metres which cannot be covered by girder bridges nor is within the economical span range of the stiffened suspension bridges. Further, as in stiffened suspension bridges, no staging or false work is required for the construction of cable stayed bridges.

17.7.2 The fundamental difference between a cable-stayed bridge and a suspension bridge is that while all the cables from the deck of a cable-stayed bridge are connected to the main tower by taut and inclined but straight cables, the twin main cables from the tower of a suspension bridge form a catenary from which the hangers are suspended and the deck system is fixed to these hangers (Fig. 17.9).

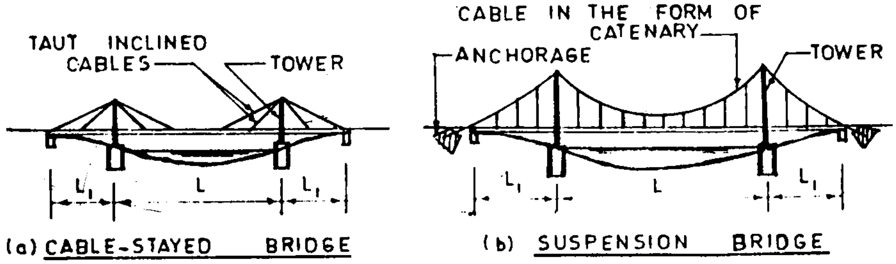


FIG.17·9 CABLE-STAYED VS SUSPENSION BRIDGE

17.7.3 The inclined taut cables of a cable-stayed bridge are relatively stiff than the cables of a suspension bridge which are relatively flexible for which the cables of a cable-stayed bridge act as intermediate elastic supports in addition to the abutment or tower support. This is not so in case of cables for suspension bridges and due to flexibility of the main cables, the support action is very small: The presence of intermediate elastic supports in a cable-stayed bridge reduces the deflection of the bridge deck as well as depth of the deck girders.

17.7.4 In cable-stayed bridges, the cables are in tension and the towers as well as the deck are in compression. By this structural system, the cable-stayed bridges offer high resistance against aerodynamic instability and as such dynamic instability has not been a problem in cable-stayed bridge. This aspect is very predominant in suspension bridges and nil in girder type bridges. Therefore, cable-stayed bridges occupy a middle position between the girder type bridges and suspension bridges in regard to aerodynamic instability.

17.7.5 The horizontal components of the cable forces from the main and the side spans balance each other while the vertical components support the vertical loads (D.L. + L.L.) of the bridge decks (Fig. 17.10). These horizontal components of the cable forces produce some sort of prestressing effect in the deck whether orthotropic steel deck or reinforced concrete composite deck and therefore, increase the load carrying capacity of the deck.

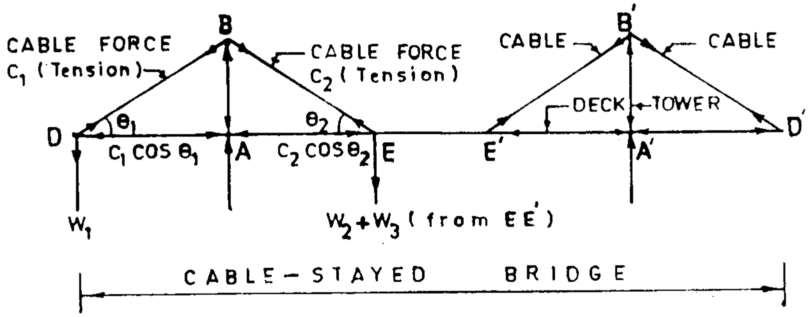


FIG. 17·10 FORCES IN A CABLE STAYED BRIDGE

In Fig. 17.10, AB is the tower and DB, BE are side span and main span cables respectively. DA and AE are the side span and main span deck. At B, horizontal components of the cable forces C_1 and C_2 balance i.e. $C_1 \cos\theta_1 = C_2 \cos\theta_2$. Similarly at A, the horizontal force in the deck due to horizontal components of the cable forces C_1 and C_2 are $C_1 \cos\theta_1$ and $C_2 \cos\theta_2$ which also balance. This horizontal force in the deck produce the prestressing effect. The vertical components of the cable forces at D and E balance the deck loads i.e., $C_1 \sin\theta_1 = W_1$ and $C_2 \sin\theta_2 = W_2 + W_3$. If $C_1 \sin\theta_1$ is greater than deck load W_1, then the end D has to be anchored such that the anchorage force F_1 is given by $C_1 \sin\theta_1 = (W_1 + F_1)$. The compression in tower AB $= C_1 \sin\theta_1 + C_2 \sin\theta_2$. Reaction at A $= C_1 \sin\theta_1 + C_2 \sin\theta_2 + W_1 + W_2$ (W_1 & W_2 are reactions from span DA & AE respectively).

17.7.6 The orthotropic steel deck with its stiffened plate or the reinforced concrete composite deck acts not only as the top flange of the main and cross girders but act also as the horizontal girder against wind forces rendering more lateral stiffness than the wind bracings used in old bridges.

17.7.7 Main towers used in cable-stayed bridges may be a single tower, A – frame, twin-towers or a portal as shown in Fig. 17.11.

17.7.8 The deck girders may consist of plate girders having orthotropic steel deck top flange and built-up bottom flange. These decks possess less torsional resistance and as such box-sections are generally used as deck girders. Box-sections may be single or twin and again may be either rectangular or trapezoidal as shown

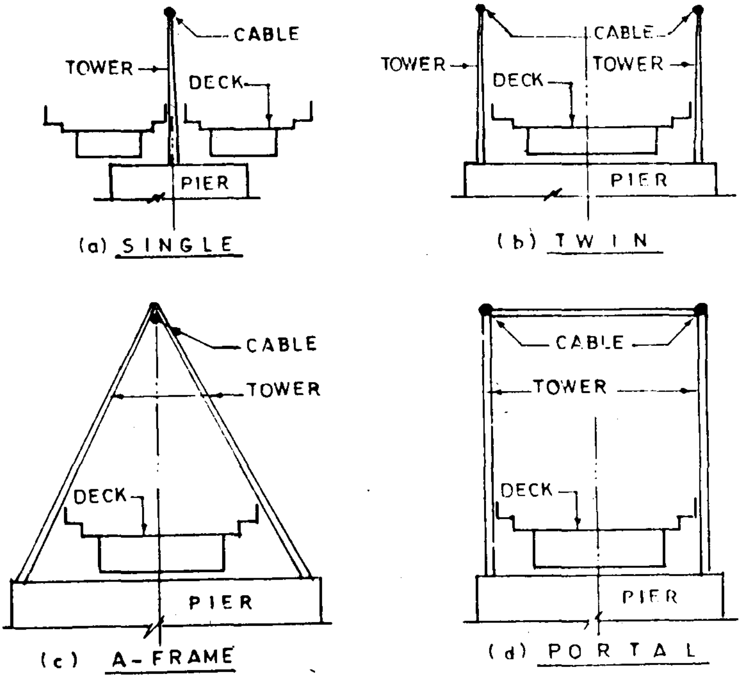


FIG. 17·11 VARIOUS TYPES OF TOWERS

in Fig. 17.12. These sections are better suited to withstand torsional moments caused by eccentric live loads. or wind forces.

17.7.9 The arrangement of the cables from the main tower to the deck varies. In *'fan'* type, the cables originate from the same point of the tower as shown in fig. 17.13a. The other types are *'harp'* type or *'modified harp'* type as in Fig. 17.13b or 17.13c. In both the harp types, only pairs of cable originate from the same point of the tower and as such there are few originating points for the cables. The difference

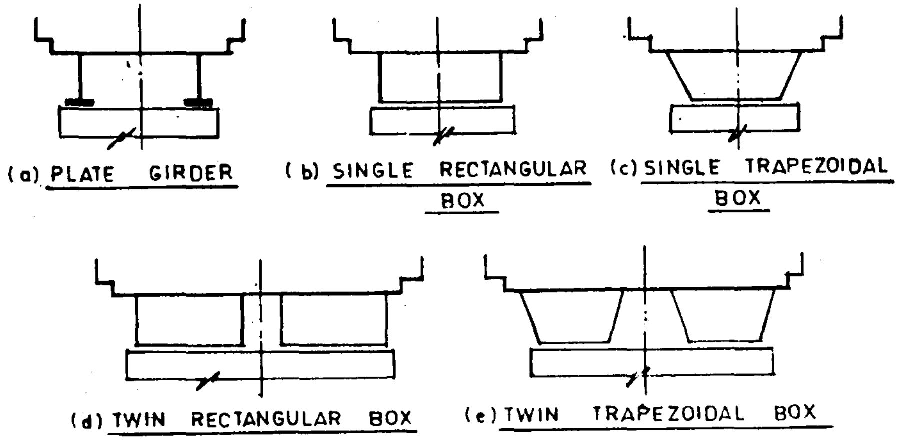


FIG.17·12 VARIOUS DECK CROSS-SECTIONS

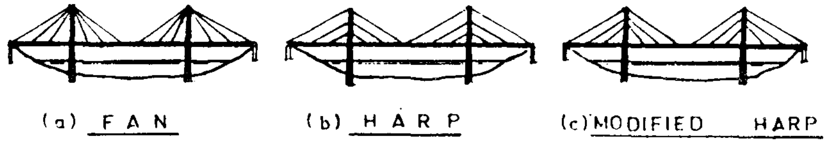


FIG. 17·13 CABLE ARRANGEMENTS FOR CABLE-STAYED BRIDGES

between the harp type and modified harp type is that in the former, the cables are all parallel having the same inclination but in the latter, the cable inclinations vary as in fan type. The cable slopes vary from $\tan\theta = 0.30$ to 0.50. Instead of single or bi-cables, multiple cables are preferable since in the last case, the cable forces are distributed at a number of points in the deck in place of one or two locations for which the depth of the deck gets reduced.

17.7.10 SALIENT FEATURES OF SOME CABLE-STAYED BRIDGES

North Bridge at Dusseldorf

This bridge was opened to traffic in 1958. The span arrangement is shown in Fig. 17.14. Twin towers as in Fig. 17.11b and two planes of cables have been used in the bridge. The deck is supported on two main box section girders 3.125 m deep × 1.60 m wide to which the cables from the towers are anchored. The spacing of the box-girders is 9.10 m. Orthotropic steel deck with 14 mm thick plate stiffened with 200 × 99 × 10 mm angles at 400 mm spacing has beam adopted. The carriageway for the bridge is 15.0 metres with 3.53 m cycle track and 2.23 m footpath. The middle cables are fixed to the towers but the top and bottom cables are placed over rocker bearings which in turn are attached to the towers.

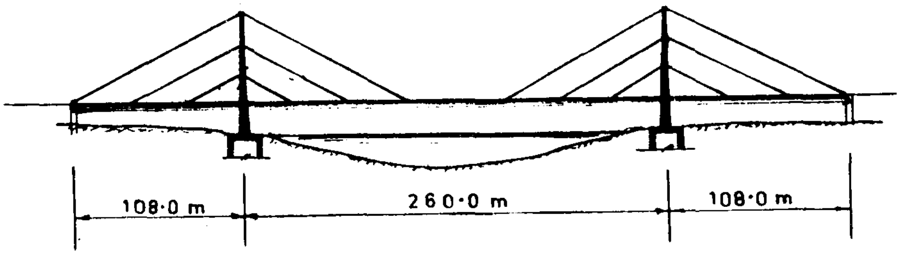


FIG. 17·14 NORTH BRIDGE AT DUSSELDORF (WEST GERMANY) (O' Connor)[6]

Bridge over the Rhine near Leverkusen, West Germany

This bridge was completed in 1965. The towers and the cables are in line with the center of the bridge deck as in Fig. 17.11a and pass through the 3.67 m wide median. Orthotropic steel deck with 61 mm thick wearing coarse supported on two-cell box-girder has been used. Extended cross girders support part of the bridge deck and the footpath (Fig. 17.15b). The bridge provides for dual carriageway of 13.0 m width separated by a 3.67 m wide central median and has 3.22 m footpath on outer side of each carriageway. The lower cables are fixed to the towers while the upper cables are placed over a rocker bearing at the top of the tower.

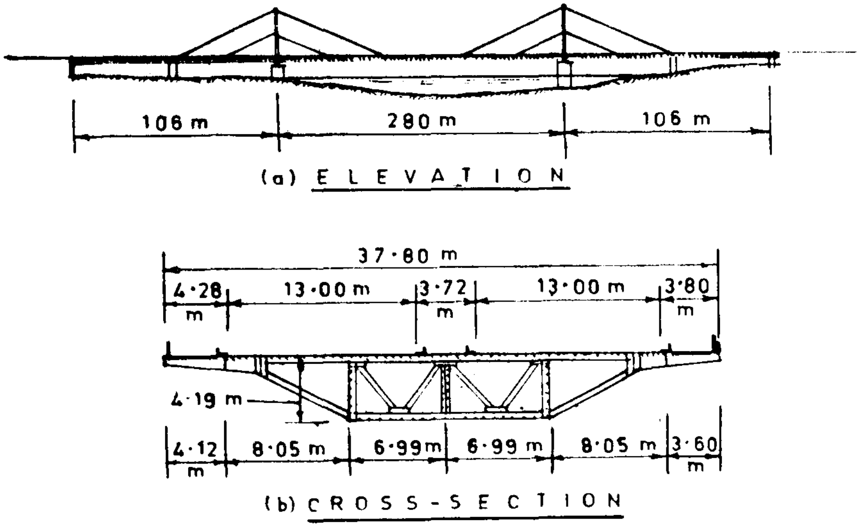


FIG.17·15 BRIDGE OVER THE RHINE NEAR LEVERKUSEN (WEST GERMANY) (O'Connor)[6]

Maracaibo Bridge, over Lake Maracaibo, Venezuela

This cable-stayed bridge completed in 1962 has seven spans viz. two ends spans of 160 metres and five intermediate spans of 235 metres (Fig. 17.16). The deck and the girders are of prestressed concrete. The cantilever portion is of three-cell box-girder section (Fig. 17.16b) while the suspended

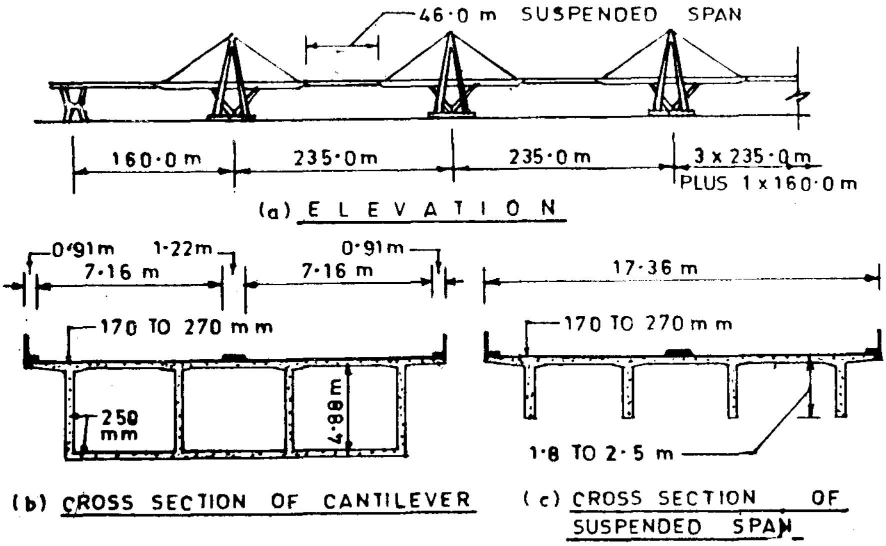


FIG. 17·16 MARACAIBO BRIDGE (VENEZUELA) (O'Connor)[6]

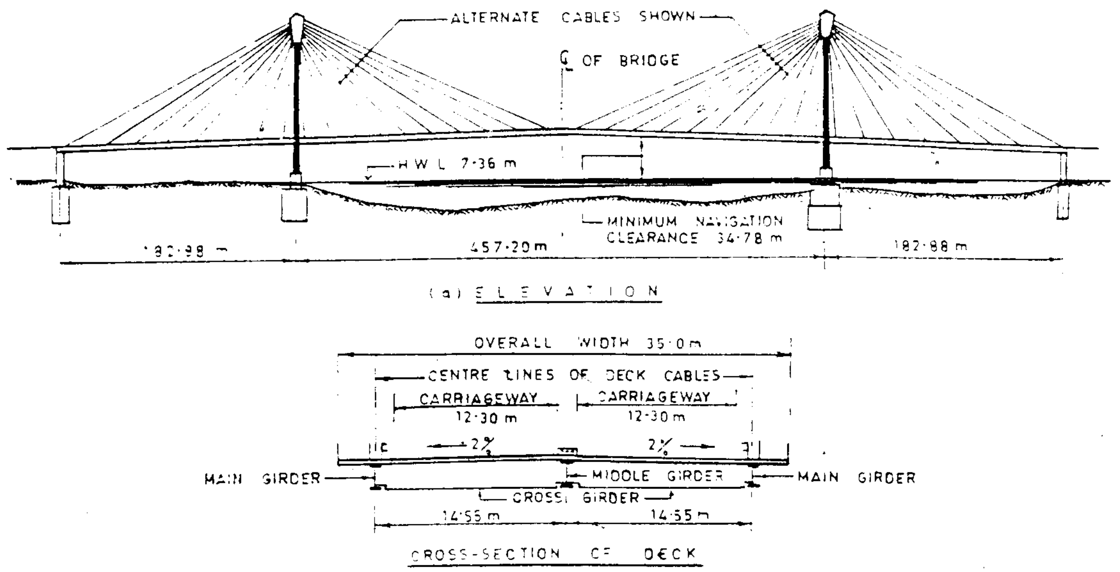


FIG. 17·17 SECOND HOOGHLY BRIDGE AT CALCUTTA (Under construction) (Ghosh et al)[3]

span has four prestressed concrete T-beam having a variable depth of 1.80 m at ends and 2.51 m at midspan (Fig. 17.16c). The bridge has a dual carriage way of 7.16 m with a central medium of 1.22 m and two footpaths of 0.91 m (fig. 17.16b). The deck slab thickness for the entire bridge varies from 170 mm to 270 mm.

Second Hooghly Bridge, Calcutta *(Under Construction)*

The span arrangement of the bridge and cross-section of the deck are shown in Fig. 17.17. The cables are in fan type arrangement as in Fig. 17.13a, total number of cable being 152. The bridge deck is a composite deck consisting of reinforced concrete deck slab supported on two main and one central steel built-up I-section.

17.7.11 Brief particulars of some more cable-stayed bridges are included in Table 17.5.

TABLE 17.5 : BRIEF PARTICULARS OF SOME MORE CABLE-STAYED BRIDGES

Sl. No.	Name	Location	Type	Max. span (metres)
1	Poleevara Bridge	Genoa, Italy	Concrete	210
2	North Elbe Bridge	Hamburg, West Germany	Steel	172
3	Severin Bridge	Cologne, West Germany	Steel	302
4	Knie Bridge	Dusseldorf, West Germany	Steel	320

17.8 SUSPENSION BRIDGES

17.8.1 Suspension bridges are economical when the span exceeds 300 metres but suspension bridges of lesser spans have also been constructed for aesthetic and other reasons in many countries. For spans exceeding 600 meters, the stiffened suspension bridges are the only solutions to cover such larger spans.

17.8.2 Suspension bridges consist of one main span and two side spans. The ratio of side span to main span generally varies from 0.17 to 0.50 (Table 17.6). Two groups of cables run from one end of the bridge to the other passing over two towers. The ends of the cables are anchored into the ground. The bridge deck supported over stiffening truss is suspended from the cables by suspenders and hence the name *"suspension bridge"*. A suspension bridge has the following components (Fig. 17.18) viz. (a) Towers, (b) Cables, (c) Anchorages, (d) Suspenders, (e) Stiffening truss, (f) Bridge deck consisting of cross-girders, stringers, and decking proper and (f) Foundation.

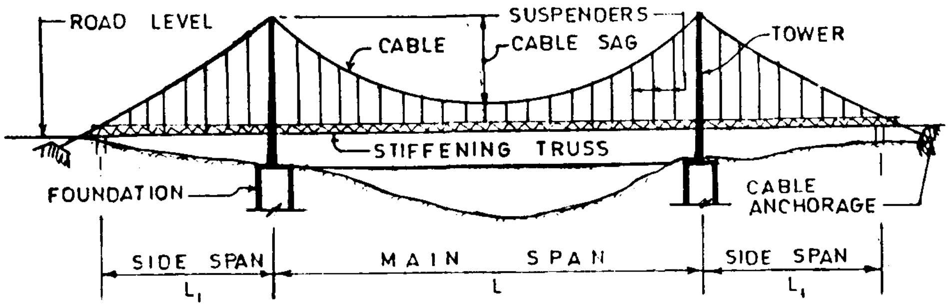


FIG. 17·18 COMPONENTS OF A SUSPENSION BRIDGE

17.8.3 The cables being very flexible do not take any bending moment and are subjected only to tensile forces. The loads from the stiffening truss are carried by the suspenders which in turn transfer the load to the cables. These cables which are subjected to tensile force transfer the loads to the towers which are regarded as sufficiently flexible and pinned at both ends. Foundations, either separate or combined, is provided below the towers for ultimate transfer of the loads to the soil strata below. The stiffening truss, as the name implies, stiffens the deck and distribute the live loads of the deck on to the cables otherwise the cables would have subjected to local sag due to action of concentrated live loads and thus caused local angle change in the deck system. The stiffening trusses are hinged at the towers and suspended at node points from suspenders which are usually high tensile cables. Vertical suspenders have been used in many bridges but diagonal suspenders as in Fig. 17.25 have the advantage that they increase the aerodynamic stability of the bridge which is very important for suspension bridges.

The cable should be cold-drawn wires and not heat-treated as the latter is susceptible to failure due to alternate stress even at low loads. The fibrous structure of the cold-drawn wires can resist alternate stresses much better than the fine-grained heat-treated wires.

17.8.4 Aerodynamic Instability

The Tacoma Narrows Bridge with a main span of 853 metres was opened to traffic on 1st July, 1940 but severaly damaged and twisted to pieces due to vertical oscillation and twisting moment caused by wind blowing at a speed of 67 Kmph. On investigation it revealed that Tacoma Narrows Bridge had a number of deviations from the conventional practices in order to have a design which would look much slender and thus be cheaper. For example, shallow plate girders were used as stiffening girder, the span-depth ratio being 350 in place of normal values of 100 to 200 (Table 17.7), span to width ratio being 72 in place of average value of 40. These changes made the deck very flexible and subjected the deck to vertical oscillation under the moving loads. On the day of the failure, a wind blowing at a speed of 67 Kmph created vertical oscillation combined with twisting motion and ultimately twisted the bridge deck to pieces.

The wind exerted on a structure causes the following forces depending upon the shape and cross-section of the deck and the angle of attack.

1. Lift and drag forces
2. Vortex formation
3. Flutter.

Flutter is the oscillation of the bridge deck in a mode including both transverse movements and torsional rotations and may occur where the natural frequencies of the two modes, taken separately, is equal to unity, i.e. $\frac{N_\theta}{N_v} = 1$, where N_θ = torsional frequency and N_v = vertical frequency. Therefore, the bridge deck must have $\frac{N_\theta}{N_v}$ values significantly greater than unity. The natural frequencies and modes of the complete structure are required to be estimated. The lowest frequencies generate (a) vertical movements with a mode at the center of the main span and (b) torsional movement with a mode also at the center of the main span. Natural frequencies of some of the existing bridges are shown in Table 17.6.

TABLE 17.6 : NATURAL FREQUENCIES OF SOME EXISTING SUSPENSION BRIDGES. *(O 'CONNOR)*[6]

Sl. No.	Name of bridge	N_θ	N_v	The ratio, $\frac{N_\theta}{N_v}$
1	First Tacoma Narrows	10.0	8.0	1.25
2	Golden Gate	11.0	5.6	1.96
3	Verrazono Narrows	11.9	6.2	1.92
4	Forth	21.1	7.6	2.78
5	Severn	30.6	7.7	3.97

17.8.5 Structural Arrangements

The following structural arrangements are made for suspension bridges :

1. Loaded or unloaded backstay.
2. Self anchored or externally anchored backstay
3. Stiffening trusses of various types
4. Various ratios of side to main span.
5. Various ratios of span to sag of cable.
6. Various ratios of span to depth of stiffening truss.
7. Tower arrangement
8. Hanger arrangement.

17.8.6 Cable Sag

The cable sag would influence the design of a suspension bridge considerably as a smaller cable sag increases the cable tension but reduces the height of towers and lengths of hangers. Therefore, where the unit cost of towers and hangers is more or where the unit cost of cables is less, smaller cable sag may be adopted and the vice versa. A reduced cable sag also increases the cable stiffness as well as the total stiffness of the structure resulting in higher natural frequency and less tendency to aerodynamic instability.

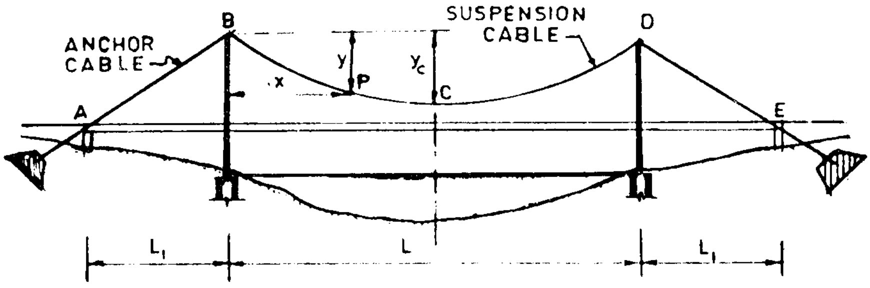


FIG. 17·19 DIMENSIONS OF SUSPENSION BRIDGES

17.8.7 Equation of the Suspension Cable

Consider a point P on the cable having co-ordinates x and y with B as the origin (Fig. 17.19). The suspension cable hangs in the shape of a parabola the equation of which is given by,

$$y = kx\,(L - x) \tag{17.1}$$

Where k = constant
L = main span.

When $x = \frac{L}{2}$, $y = y_c$ (the cable sag at center of main span).

Substituting these values, equation 17.1 becomes, $y_c = k \cdot \frac{L}{2}(L - \frac{L}{2}) = \frac{kL^2}{4}$ $\therefore$ $k = \frac{4y_c}{L^2}$

Putting the value of k in equation 17.1,

$$y = \frac{4y_c}{L^2}\,x\,(L - x) \tag{17.2}$$

Equation 17.2 gives the dip y of the cable from its tower support at any distance x from B.

17.8.8 Tension in the Cable.

From Fig. 17.20, vertical reaction on tower due to load ω per unit length = $R_B = R_D = \frac{\omega L}{2} = R$.

The cable being flexible, cannot take any moment and as such the moment at mid span of the cable is zero. Therefore, taking moment of the left hand side loads and forces about C,

$$R_B \cdot \frac{L}{2} = H \cdot y_c + \omega \cdot \frac{L}{2} \cdot \frac{L}{4} \quad \text{or,} \quad H \cdot y_c = \frac{\omega L^2}{8}$$

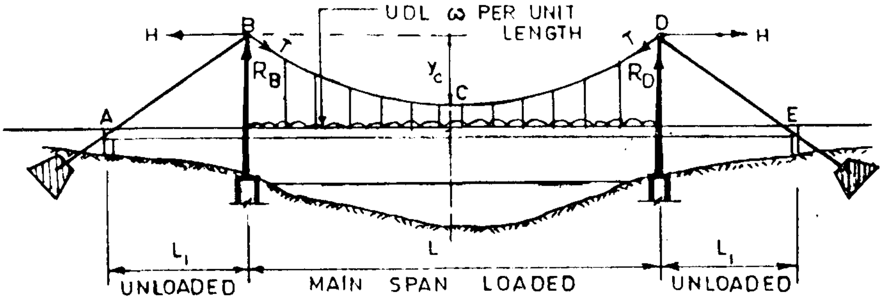


FIG.17·20 CABLE TENSION IN A SUSPENSION BRIDGE

$$\text{or,} \quad H = \frac{\omega L^2}{8\, y_c} \tag{17.3}$$

Maximum tension in the cable at B,

$$T = \sqrt{H^2 + R^2} \tag{17.4}$$

17.8.9 Back-Stay Cables

The suspension cable of the main span is supported on two towers on both sides of the main span. The suspension cable after passing over the supporting tower is generally anchored down into a mass of concrete or some sort of anchoring arrangement. The cable of the side span is termed *"anchor cable"* or *"back-stay"* cable. The following two arrangements are made for passing the cables over the towers from the main span to the side span :

1. Guide pulley support
2. Roller support.

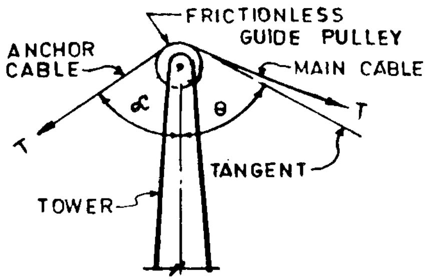


FIG.17·21 GUIDE PULLEY SUPPORT FOR SUSPENSION CABLE (Khurmi)[5]

Guide Pulley Support for Suspension Cable

The main cable is taken over a frictionless guide pulley fixed on the top of the supporting tower to the side span and then anchored. In Fig. 17.21, α and θ are the angles the cables make with the center line of the tower and T is the tension in the cable. Since the cable passes over a frictionless pulley, T on both sides is the same.

Vertical reaction on the tower due to cable tension,

$$R_T = T\cos\alpha + T\cos\theta \tag{17.5}$$

Horizontal force on the top of the tower,

$$T\sin\alpha - T\sin\theta = T(\sin\alpha - \sin\theta) \tag{17.6}$$

Roller Support for Suspension Cable

In this arrangement of supporting cables, both the main cable and the anchor cable are attached to a saddle which is supported on rollers placed at the top of the tower (Fig. 17.22).

Since the saddle is at rest, the horizontal components of both the main and the anchor cables must be the same, i.e.,

$$H = T_1 \sin\alpha = T_2 \sin\theta \qquad (17.7)$$

Vertical reaction on the tower due to tension in the cables,

$$R_T = T_1 \cos\alpha + T_2 \cos\theta \qquad (17.8)$$

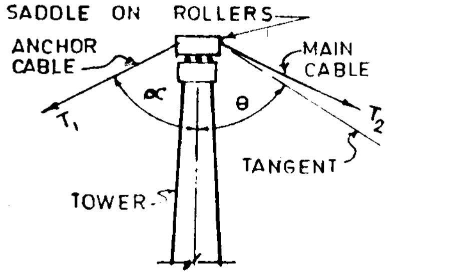


FIG.17·22 ROLLER SUPPORT FOR SUSPENSION CABLE (Khurmi)[5]

ILLUSTRATIVE EXAMPLE 17.1

A suspension bridge having a main span of 100 metres has a cable sag of 10 metres. Calculate the maximum tension in the cables when the deck is carrying a load of 50 KN per metre length. Also find the vertical reaction on the tower (a) if the cable passes over a friction less pulley and (b) if the cable passes over a saddle resting on rollers.

Given :	L	=	*main span = 100 m*
	y_c	=	*cable sag at center = 10 m*
	w	=	*UDL = 50 KN per m.*
	α	=	*angle of anchor cable = 60°*

Solution

Case (a) The cable passes over a frictionless pulley

Vertical reaction on tower due to load w, $R = \dfrac{wL}{2} = \dfrac{50 \times 100}{2} = 2{,}500$ KN

From equation 17.3, horizontal pull on the cables, $H = \dfrac{wL^2}{8\,y_c} = \dfrac{50 \times (100)^2}{8 \times 10} = 6250$ KN

Maximum tension in the cable from equation 17.4, $T = \sqrt{H^2 + R^2} = \sqrt{(6250)^2 + (2500)^2} = 6730$ KN

Vertical load on top of tower from equation 17.5, $R_T = T\cos\alpha + T\cos\theta$

$= T\cos\alpha + R$ (i.e., reaction due load w = 50 KN/m)

$= 6730 \cos 60° + 2500 = 3365 + 2500 = 5865$ KN

Case (b) The cable passes over a saddle on rollers

H as in case (a) = 6250 KN. From equation 17.7, $H = T_1 \sin\alpha$. $\therefore\ T_1 = \dfrac{H}{\sin\alpha} = \dfrac{6250}{\sin 60°} = 7220$ KN

Vertical load on tower, R_T from equation 17.8 $= T_1\cos\alpha + T_2\cos\theta$

$= T_1\cos\alpha + R$ (i.e. reaction due load w=50 KN/m)

$= 7220 \cos 60° + 2500 = 3610 + 2500 \quad 6110$ KN

17.8.10 BRIEF DESCRIPTION OF SOME EXISTING SUSPENSION BRIDGES

Forth Road Bridge (Scotland)

The elevation of the bridge is shown in Fig. 17.23. The main span has an orthotropic steel plate deck with 38 mm thick asphaltic wearing surface. The side spans have 222 mm. thick concrete slab with a wearing surface of 38 mm thick asphaltic concrete as in main span. The span depth ratio of the stiffening truss is 120. Some more features are shown in Table 17.7.

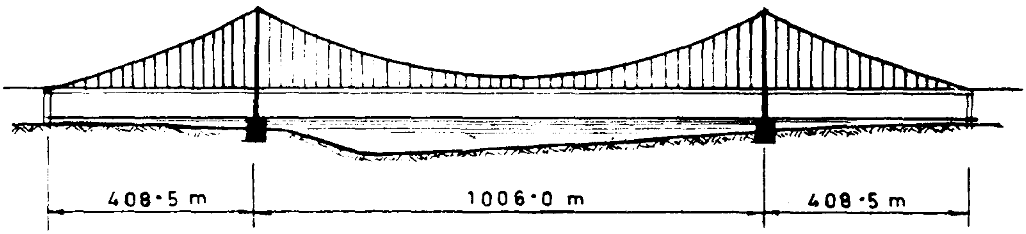


FIG. 17·23 FORTH ROAD BRIDGE (SCOTLAND)
(O' Connor)[6]

Mackinac Bridge (U.S.A.)

The elevation of the bridge is shown in Fig. 17.24. The bridge provides for a four lanes carriageway carried on 108 mm. thick steel grating. While the outer lanes are covered with concrete, the central dual carriageway is left open from aerodynamic consideration. The span-depth ratio of the stiffening truss in Mackinac Bridge is 100. Some more features of the bridge are shown in Table 17.7.

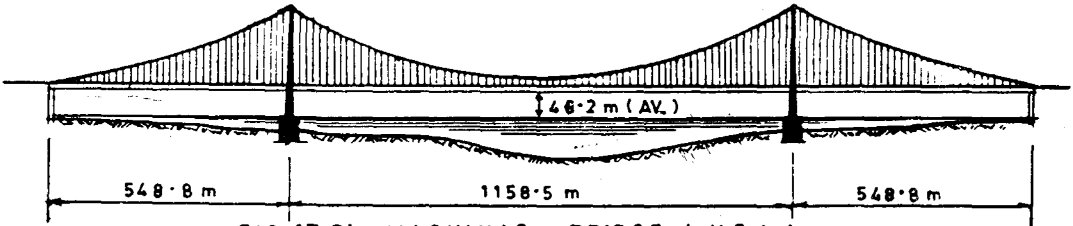


FIG. 17·24 MACKINAC BRIDGE (USA)
(O' Connor)[6]

Severn Bridge (Wales)

The elevation of severn bridge is shown in fig. 17.25. The bridge has a dual carriage way of 9.91 m each. Instead of stiffening truss, tubular or box-girder steel section of aerofoil design has been used in the bridge.

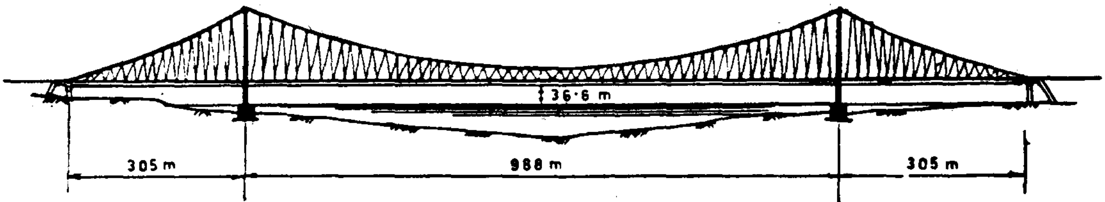


FIG. 17·25 SEVERN BRIDGE (WALES) (O'Connor)[6]

TABLE 17.7 : SALIENT FEATURES OF SOME SUSPENSION BRIDGES (O Connor)[6]

Sl. No.	Name of Bridge	Location	Year of Opening	Nos. of Traffic Lanes	Span (metres)		Ratio of Side Span to Main Span	Central Clearance (m)	Tower Height (m)
					Main	Side			
1	Golden Gate	USA	1937	6	1280	343	0.27	67.00	227.4
2	First Tacoma Narrows	USA	1940	2	854	335	0.39	59.76	132.3
3	Second Tacoma Narrows	USA	1950	4	854	335	0.39	56.10	153.0
4	Mackinac	USA	1958	4	1158	549	0.47	64.94	181.4
5	Forth	Scotland	1964	4	1006	409	0.41	51.83	156.1
6	Verrazano Narrows	USA	1964	12	1299	370	0.28	69.51	210.4
7	Severn	Wales	1966	4	988	305	0.31	36.58	135.7

TABLE 17.7 (CONTD.) : SALIENT FEATURES OF SOME SUSPENSION BRIDGES

Sl. No.	Name of Bridge	Cable Sag (m)	Ratio of Main Span to Cable Sag	Spacing of Cables (m)	Spacing of Hangers (m)	Type of Stiffening System	Depth of Stiffening Girder/ System	Ratio of Main Span to Depth of Stiffening Girder/ Truss	Type of Deck
1	Golden Gate	144.82	8.84	27.44	15.24	Truss	7.62	168	Concrete
2	First Tacoma Narrows	70.73	12.07	11.89	15.24	Plate Girder	2.44	350	Concrete
3	Second Tacoma Narrows	87.20	9.79	18.29	15.24	Truss	10.06	85	Concrete
4	Mackinac	96.34	12.02	20.73	11.89	Truss	11.59	100	Concrete Filled Grid
5	Forth	91.46	11.00	23.78	18.29	Truss	8.38	120	Orthotropic Steel
6	Verrazano Narrows	117.38	11.07	31.40	15.09	Truss	7.32	177	Concrete Filled Grid
7	Severn	82.32	12.00	22.87	18.29	Box Girder	3.05	324	Orthotropic Steel

The traffic is carried directly by a 11.5 mm. thick stiffened steel plate. The special feature of this bridge is not only the tubular section instead of stiffening truss but also the inclined hangers in place of vertical hangers. The hanger spacing is 18.3 metres and the inclination of the hanger with the vertical varies from 17.5 degrees to 25 degrees. Some additional features are shown in Table 17.7.

Verrazano Narrows Bridge (USA)

The elevation of the bridge is shown in fig. 17.26. The bridge has double decks with 6 lanes carriageway in each deck. In each deck, three lanes dual carriageway having a central median of 1.22 m and carriage way width of 11.28 m have been provided. The span-depth ratio of the stiffening truss is 177.5 and the center to center of main cables is 31.4 m. Some more features of the bridge are shown in Table 17.7.

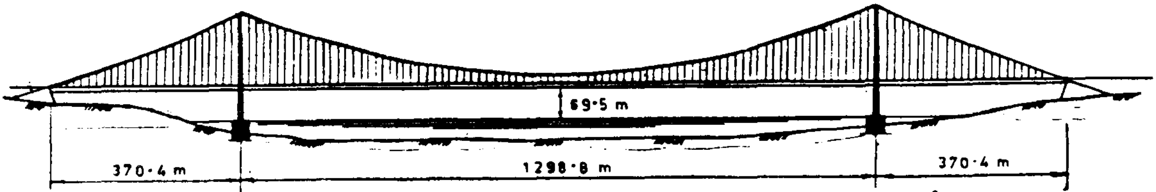


FIG. 17·26 VERRAZANO NARROWS BRIDGE (USA) (O'Connor)[6]

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CHAPTER 18

TEMPORARY, LOW COST AND MOVABLE BRIDGES

18.1 TEMPORARY BRIDGES

18.1.1 General

18.1.1.1 Temporary bridges are used in forest roads where timber is found in plenty at cheaper cost. These are also used in less important roads where the traffic is less and the loading is light. Due to gradual development of the area adjacent to the road when the growth of traffic is more as well as the loading of the vehicles increases, these temporary bridges are no longer capable of carrying these increased frequency and increased loading of vehicles and therefore, are replaced by permanent bridges. Sometimes, shortage of fund dictates the construction of temporary bridges which are converted to permanent bridges when fund permits. Sometimes, temporary bridges are constructed on important roads as fair-weather bridges on major river crossings. During monsoon, ferry service transports the vehicles but as soon as the monsoon is over, fair-weather bridges are constructed to carry vehicular traffic till the next monsoon when these temporary bridges are dismantled and ferry service is again resumed.

18.1.1.2 Temporary bridges are widely used during war when a channel or a river has to be crossed by the military for the movement of troops, ammunitions and food supply. The temporary bridge may be over a crossing where no bridge exists or may be near an existing site where the bridge has been destroyed by the enemy. To maintain the existing traffic, temporary bridges are essentially required to be constructed on a diversion, when the existing bridge has to be replaced by a new one for any reason what so-ever. Temporary bridges also give very useful service during floods which sometimes cause breach in the road embankment. In such emergent situation, building of the road embankment with earth is not possible as no earth is available nearby due to flood water and as such bridging the gap with some temporary bridge is the only answer.

18.1.1.3 Timber is extensively used in the construction of temporary bridges when such construction is to be undertaken as per planned programme and necessary time is available for this purpose. Temporary bridges on less important road or on a diversion during replacement of an existing bridge or as fair-weather bridges are usually of timber construction. Emergency temporary bridges are required to be built without pre-planning and therefore, prefabricated units are necessary for erection of the bridge very quickly with short notice. Bailey bridge, Inglis bridge and Callender Hamilton bridge are such type of bridges. All these bridges were extensively used by the army engineers during the Second World War.

18.1.2 Timber Bridges

18.1.2.1 Timber bridges are generally constructed on salbullah pile trestle as sub-structure and foundation and timber decking over timber beams or over RSJ as superstructure. The pile trestle is composed of a cluster of piles usually 200 mm. diameter driven to 3.0 m for 3.0 m span bridges and 4.5 to 6.0 m for 4.5 m and 6.0 m span bridges. These driven depths shall be below the bed level taking into consideration of the possible scouring of the bed if any. The piles may be in single row for 3.0 m span as in Fig. 18.1 and double rows for 4.5 m and 6.0 m span as in Fig. 18.2. Timber beams are used for 3.0 m span superstructure and these beams are seated on single pile trestle by m.s. angle cleats and m.s. plates. When RSJ used over double rows of pile trestle for longer spans, one m.s. channel is placed on the top of the piles and RSJ are seated on these channels. While the decking planks are fixed on to the timber beams directly by nails, an intermediate packing timber piece is fixed by countersunk bolts over the RSJ for the facility of nailing the decking timber over this timber piece. Wooden sleepers are used as wheel guards on both sides of the carriageway. Two trackways of 50 mm. thick timber planks are longitudinally placed on both sides of the center line of the transverse deck for distribution of live loads (wheel loads) over a few decking planks. 50 mm × 50 mm deck stiffeners are also fixed at the bottom of decking planks for the same purpose.

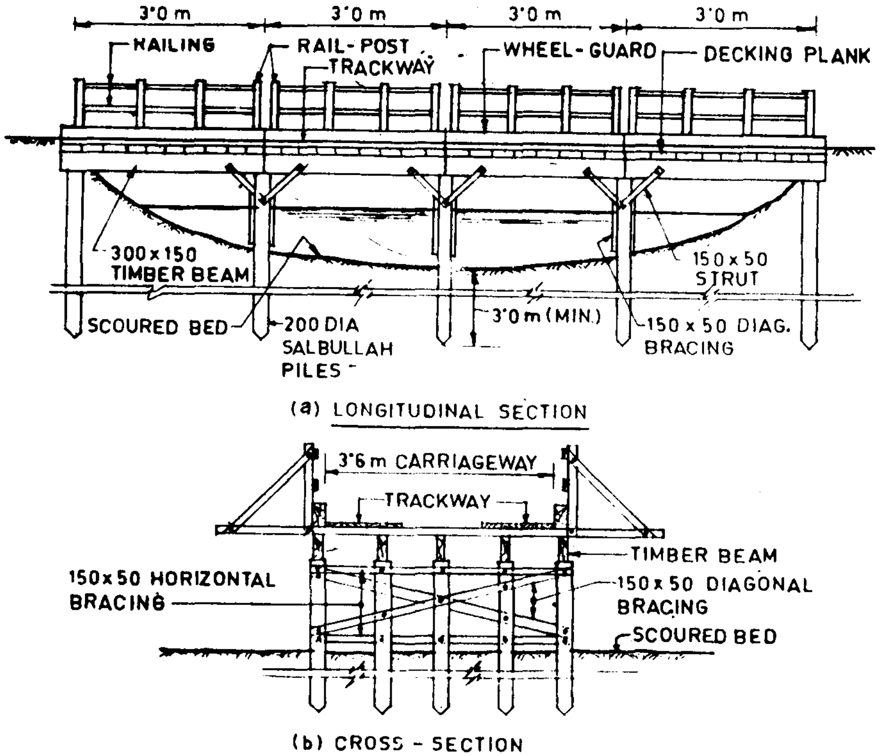


FIG. 18·1 TIMBER BRIDGE WITH TIMBER BEAMS

18.1.2.2 The timbers used for bridge works shall be seasoned and treated with preservatives such as ASCU or creosote oil. The salbullah piles shall be painted with two coats of hot coal tar before driving. The design shall be based on safe working stresses for tension, compression and bending as given in Table 18.1 for some Indian timbers commonly used.

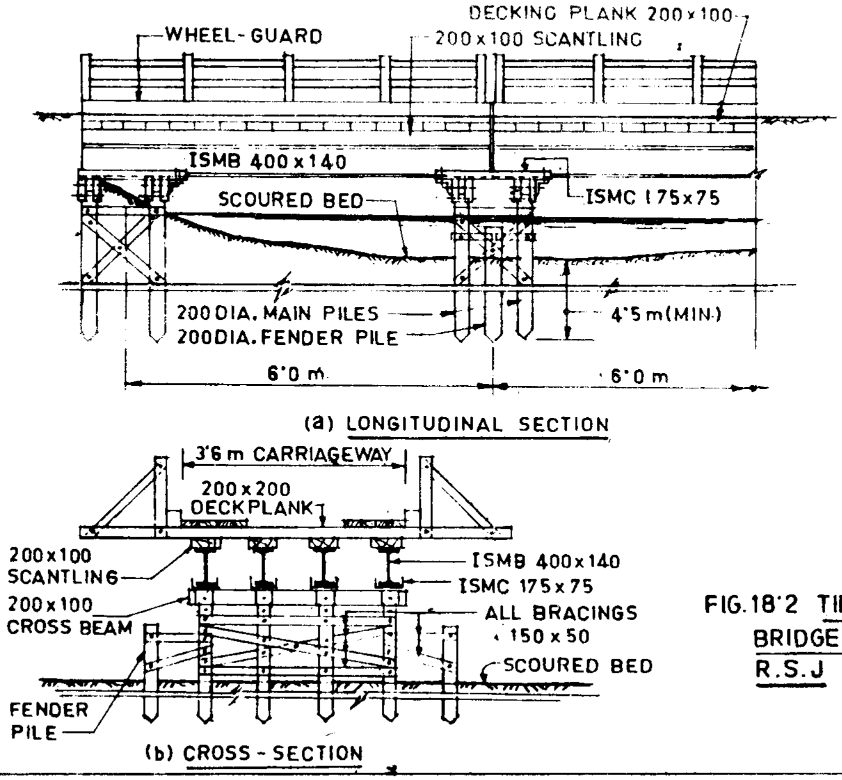


FIG.18'2 TIMBER BRIDGE WITH R.S.J

TABLE 18.1 : SAFE WORKING STRESSES FOR INDIAN TIMBERS (Khanna)[7]

Trade Name	Average weight in Kg./m³	Mod. of Elasticity in T/Sq. cm	Bending & tension along grain (Kg/sq.cm)		Shear Stress (Kg/sq.cm)		Comp. Stress (Kg/sq.cm.) Parallel to grains		Comp. Stress (Kg/sq.cm.) Perp. to grains	
			Outside location	Wet location	Horizontal	Along grain	Outside location	Wet location	Outside location	Wet location
Babul/ Kikar	835	108	154	124	15.4	22.4	102	80	50.5	41.5
Benteak	675	110	112	92	9.2	13.0	78	64	32.5	26.0
Blue pine	515	68	56	50	5.6	8.0	46	38	13.5	10.5
Chir	575	98	70	60	6.4	9.2	56	46	17.5	14.0
Deodar	560	95	88	70	7.0	10.2	70	56	21.0	17.0
Fir, Partal	465	94	66	56	6.0	8.4	52	42	12.5	10.5
Haldu	675	91	112	92	9.4	13.4	74	64	28.0	23.0
Kail	515	68	56	50	5.6	8.0	46	38	13.5	10.5
Sal	800	127	140	112	9.4	13.4	94	78	35.0	29.0
Spruce	480	92	66	52	6.0	8.4	50	42	13.5	10.5
Teak	625	96	116	94	9.8	14.0	78	64	31.0	25.5
Walnut	575	91	94	78	8.4	12.0	60	50	18.5	15.0

18.1.3 Bailey Bridge

18.1.3.1 Bailey bridges were designed during the Second World War and extensively used in military operation as temporary bridges. These are still available as Portable Steel Bridges manufactured by Garden Reach Shipbuilders & Engineers Ltd., Calcutta with some modification. These bridges are also used as semi-permanent and permanent bridges. This is a steel truss bridge of *'through'* type having steel panels of 3050 mm. long and 1450 mm high. These panels are of welded construction and are pin-joined to adjacent panels. As the span increases, the panels are arranged both side by side or one above the other to increase the

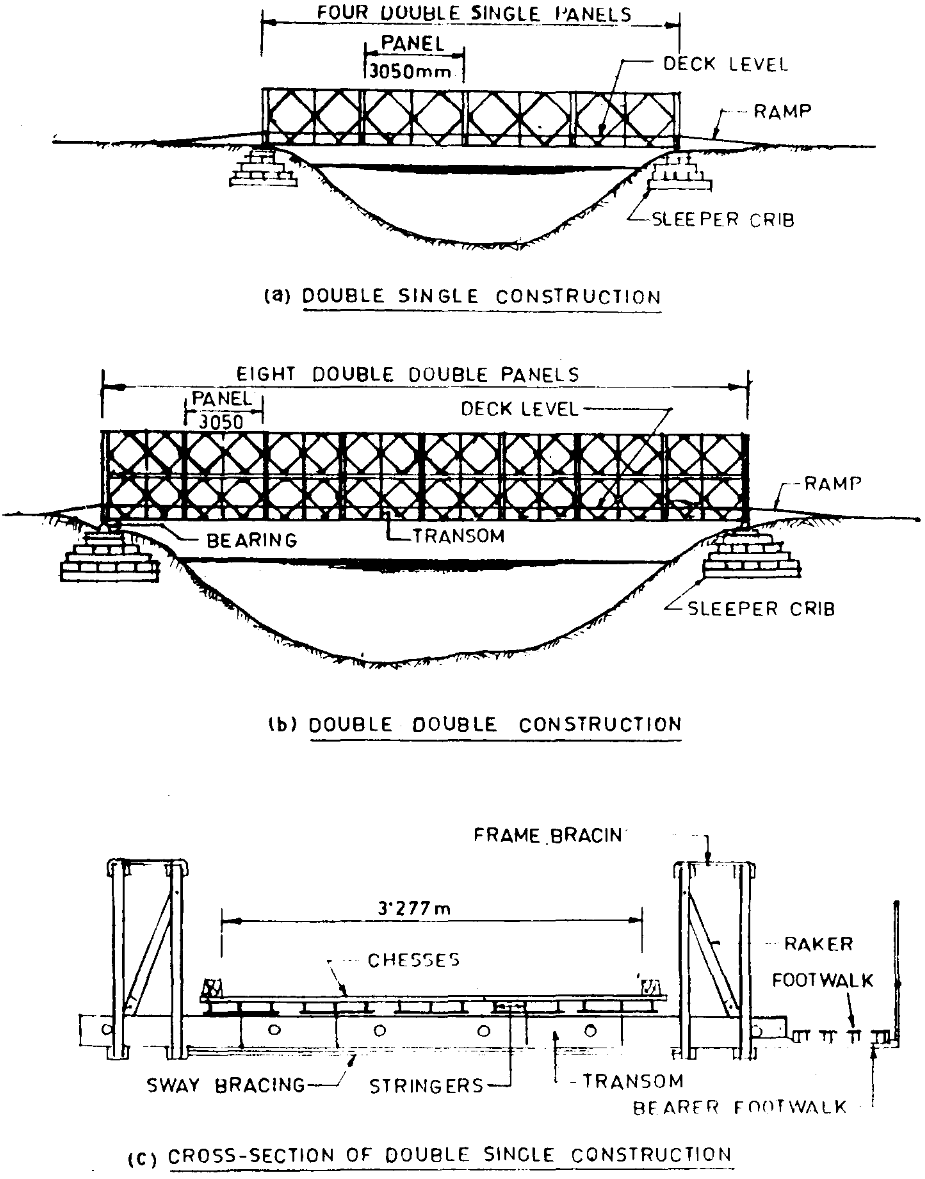


FIG. 18·3 PORTABLE STEEL BRIDGES (BAILEY BRIDGES) (GRSE)[1] load carrying capacity. The bridge is designated according to the number of trusses and storeys by which the bridge girders are formed. In such designation, the first word indicates number of trusses side by side and the second word indicates number of panels one above the other. For example *'single single'* indicates single truss one storeyed. This is the lightest truss having a span of 15.2 metres. Similarly, *'tripple double'* indicates three trusses side by side having two storeys. Heaviest girder is the *'tripple tripple'* with three trusses side by side and three storeyed trusses. *'Single double'* or *'single tripple'* girder are unsafe and hence are never used. Maximum span covered by this type of bridge is 61 metres.

18.1.3.2 The load carrying capacity of Bailey bridges is designated as Class 3, 5, 9, 12, 18, 24, 30, 40, 50, 60 and 70 and corresponds to the load in tonnes approximately as the class number. A bridge of Class 3 can carry a jeep, Class 9 can carry a truck, Class 50 can carry a centurian tank moving on its own power and Class 70 can carry the same tank when carried over a trailer. These vehicle classifications have also been included in the IRC Bridge Code. Class 24, 40 and 70 correspond to IRC Class B, A and AA loading respectively.

18.1.3.3 The cross-girders or *'transoms'* are long RSJ resting on the bottom chord of the girders. The *'road-bearers'* are supported on the transoms. These road-bearers are welded together in groups of three for the facility of easy construction and placed longitudinally. The wooden *'chesses'* form the decking of the bridge. These chesses are seated over the road-brearers and as such are placed transversely.

18.1.3.4 The end posts at the end of the truss are supported on bearings. These bearings are seated over base plates which distribute the load from the bridge onto the ground. When the bearing power of the ground cannot take load from the base plate, these are placed on sleeper cribs. When these bridges are constructed on semi-permanent basis, bearing plates are fixed onto abutments.

18.1.3.5 Footwalks of 760 mm wide can be provided either on one side or on both sides for the pedestrians. These footwalks are placed beyond the main girders with the help of footwalk bearers which are fixed to the end of the transoms.

18.1.3.6 These bridges when used as temporary bridges on emergent situation, require ramps to negotiate the difference of level between the road and the deck which is about 710 mm above the bottom of base plate. Ramps of 3050 mm and 6100 mm are available. The former is used to negotiate small difference of level and the latter to negotiate large difference.

18.1.3.7 Bailey bridges may be constructed on temporary supports or may be launched in position with the help of a *'launching nose'* and some rollers. The launching nose is made from the same panels as the bridge and its length is such that when the complete bridge truss along with the launching nose is moved over a number of rollers on one bank and the launching nose is drawn to the other bank, the stability of the whole unit is maintained without any overtunning of the unit (Fig. 18.4). If required, some kentledge load may also be used to maintain stability.

18.1.3.8 When the launching nose reaches the other bank, the unit is further moved till the bridge truss ends reach the final position over the bearings. The nose and the kentledge are then removed and the bridge truss is lowered by jacks and finally seated on the bearings.

Bailey bridges can be erected even by unskilled labour under the supervision of a trained engineer. The heaviest component which weighs 300 Kg can be carried by six men. All components of a Bailey bridge can be transported to site in commercial 3-tonnes trucks.

18.1.4 Callender-Hamilton (Unit Construction) Bridge

18.1.4.1 Callender-Hamilton bridges were originally designed for the army to carry millitary loadings. This type of bridge covers a span of 12.0 m to 42.0 m and is a *'through type'* truss like Bailey bridge. Callender Hamilton bridges are also used as semi-permanent or permanent bridges. The basic member used

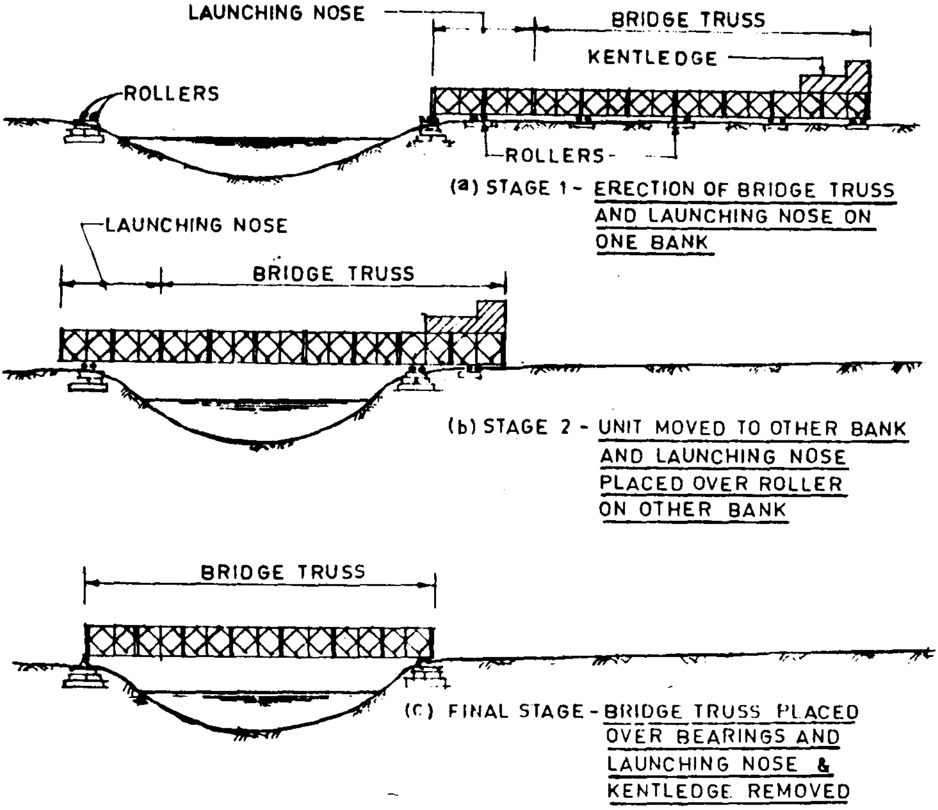


FIG. 18·4 ERECTION AND LAUNCHING OF PORTABLE STEEL BRIDGES (BAILEY BRIDGES) (GRSE)[1]

in the construction of this type of bridge is an angle 150 mm × 150 mm × 20 mm × 3.0 metres long which is used as top chord, bottom chord and diagonals of the Warren type truss as shown in Fig. 18.5.

18.1.4.2 The length of top and bottom chords is 3.0 m. and as such the bridge can be constructed in multiples of 3.0 metres. For spans from 12.0 m to 24.0 m single truss on either side is used but for spans beyond 24.0 m and upto 42.0 m, two trusses bolted together in pairs are used on either side. The load carrying capacity of those bridges varies from 18 tonnes vehicle to 30 tonnes vehicle.

18.1.4.3 The trusses support the cross-bearers which are 300 mm × 90 mm channels. The road bearers (see Fig. 18.5) seated on cross bearers are made of timber having length of 3.0 m. and cross section of 250 mm × 125 mm and fixed to the cross-bearer by 20 mm. diameter bolts. The deck planks or chesses are 3.37 metres long having cross-section of 250 mm × 100 mm. The ribands are also of timber section (225 mm × 225 mm). The wooden decking as described above can be replaced by steel troughs or by reinforced concrete provided the substitute decking does not weigh more than 3.35 tonnes per 3.0 m bay.

18.1.4.4 Callender-Hamilton bridges may be constructed by any of the following methods depending upon site condition as well as availability of skilled men and equipments :

i) Erection in situ by using temporary intermediate supports.

ii) Erection and launching from one bank to the other with the help of launching nose as described in Art. 18.1.3 for Bailey bridges.

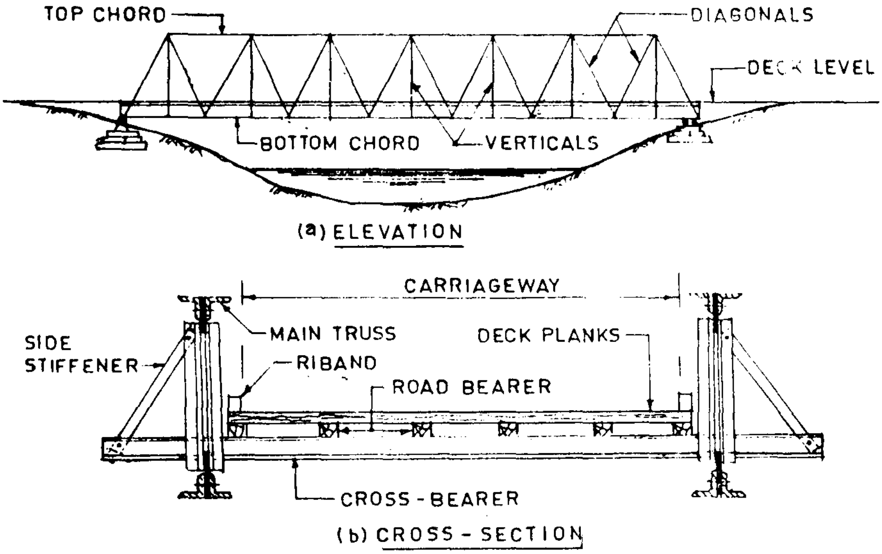


FIG. 18.5 CALLENDER-HAMILTON (UNIT CONSTRUCTION) BRIDGE (Military Engg.)[2]

iii) Erection and launching complete truss one at a time from one bank to the other by derricks, winches and anchorages (Fig. 18.6).

18.2 LOW COST BRIDGES

18.2.1 This category of bridges is constructed when sufficient funds are not available. The temporary bridges described in Art. 18.1 viz. timber bridges, Bailey bridges and Callender-Hamilton bridges also come under this category. In these bridges, the carriageway is kept limited to single lane for the sake of economy which is the prime consideration for the construction of such type of bridges for want of sufficient funds. Another two types of low cost bridges which are generally constructed are (a) Causeways and (b) Submersible bridges.

18.2.2 Causeways

18.2.2.1 Causeways are constructed under the following circumstances :

i) When the importance of the road is not much and the stream carries little or no water during dry season and small discharge during monsoon except floods.
ii) The flood discharge in the stream flows only for small duration say 24 to 72 hours.
iii) In hilly roads where small water courses cross the roads at frequent intervals.

18.2.2.2 ***Low level causeways*** have their top level flush with or little above the bed of the stream so as to pass the flood discharge over them (Fig. 18.7). During the high floods, traffic has to be suspended for 24 to 72 hours on six to eight occasions during monsoon.

The streams over which these causeways are constructed remain dry or carry very small discharge during dry season and flood discharge in the stream flows only for short duration say 24 to 72 hours.

To protect the causeway slab against scour, cut-off walls or curtain walls are constructed both on U/S and D/S sides. The depth of D/S side curtain wall is more since the scour on the D/S side is more. Generally, the curtain walls are taken to 1.5 to 2.0 metres for U/S side walls and 2.0 to 2.5 metres for D/S walls.

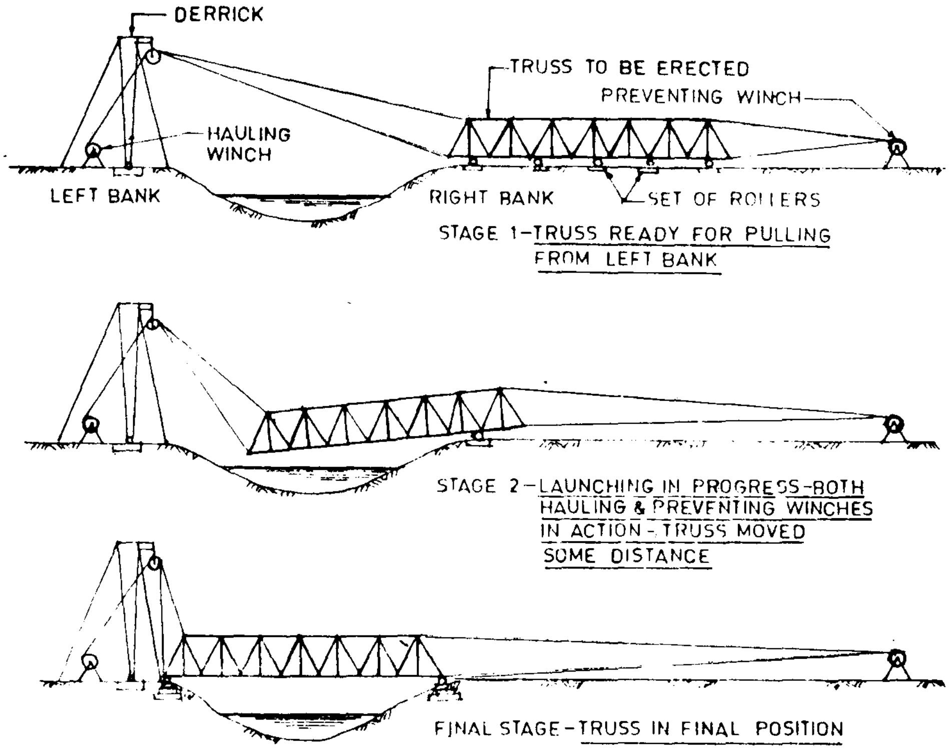


FIG. 18·6 ERECTION AND LAUNCHING OF CALLENDER-HAMILTON (UNIT CONSTRUCTION) BRIDGE (M. Engg.)[2]

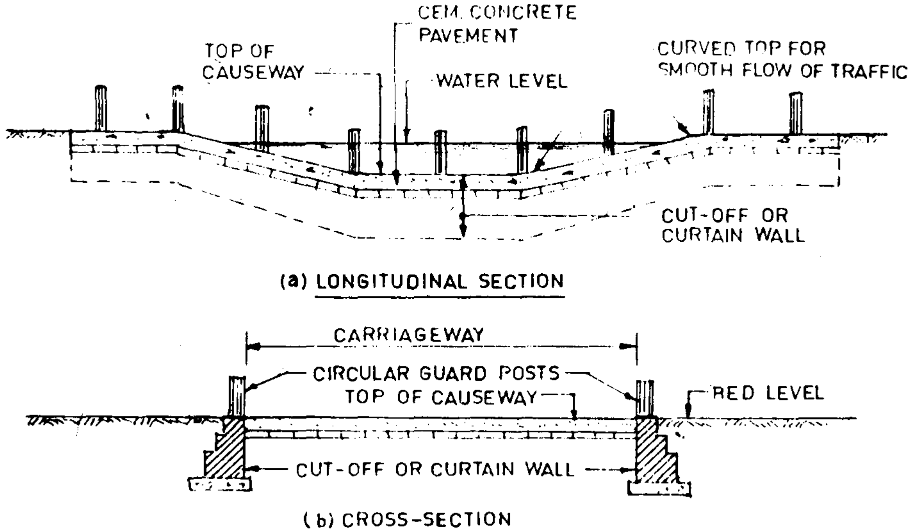


FIG. 18·7 LOW LEVEL CAUSEWAY

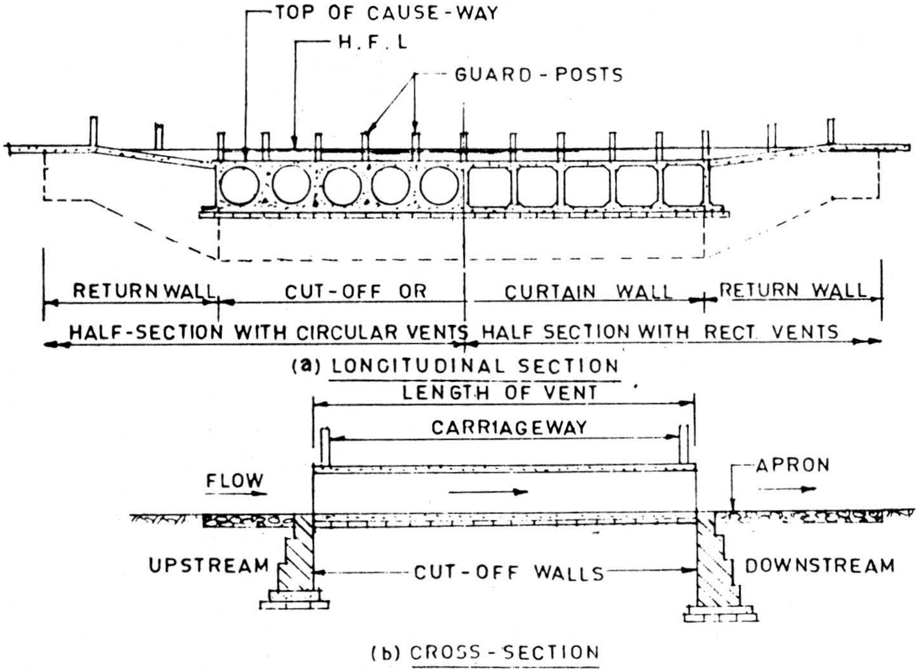
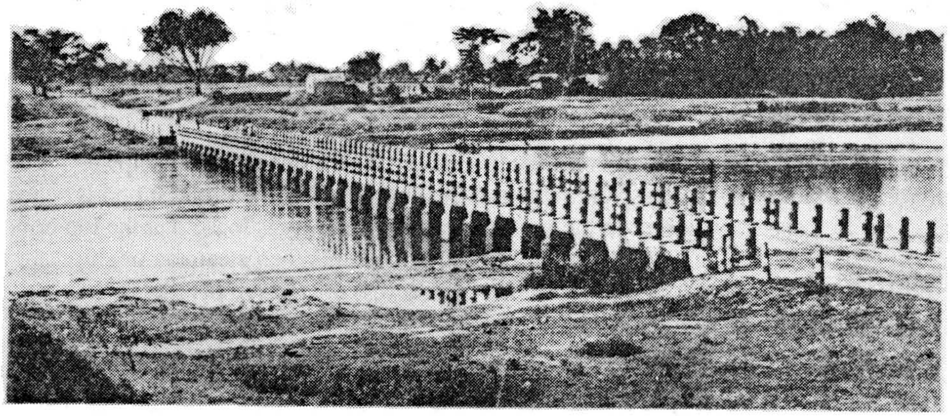


FIG. 18.8 VENTED CAUSEWAY

18.2.2.3 ***High level or vented causeways*** are constructed where water flows perennially during the whole year and during monsoon, high flood discharge passes for 24 to 72 hours for 6 to 8 occasions. The invert level of these vents is kept near the bed level and top level of the causeways is kept such that traffic is not dislocated for more than 15 to 25 days during the monsoon. The vents may be circular or rectangular as shown in Fig. 18.8a left half or right half respectively. A vented causeway with rectangular vents may be seen in Photograph 5.



Photograph 5
VENTED CAUSEWAY OVER SILABATI RIVER
(Constructed over Simlapal-Sarenga Road in the District of Bankura, West Bengal.
Consists of 37 Nos. 2.4 m × 2.4 m R. C. Vents. Total length is 102.0 m). *(Highway Bridges)*[3]

Cut-off or curtain walls are provided both on upstream and downstream sides as in low level causeway. In addition, aprons in front of these causeways are used upto the curtain walls for smooth flow of water through the vents. Usually, 2.0 to 2.5 m apron on the U/S side and 3.0 to 4.5 m apron on the D/S are provided.

18.2.3 Submersible Bridges

18.2.3.1 A submersible bridge is a compromise between the vented causeway and the high level bridge. In case of vented causeway, except normal flow, all flood water whether of ordinary floods or of high floods will pass over the vented causeway and in high level bridge, all floods including highest floods will pass underneath the bridge with some free-board from the soffit of the deck. But in submersible bridges, ordinary floods will pass below the bridge deck but high floods will pass over the deck. Therefore, for the same site, low level or flush causeway, if provided, will cause maximum traffic interruption, high level or vented causeways will create less interruption and the submersible bridges will cause least interruption to traffic. It is for this reason that while causeways are used in hill roads, village roads and other district roads, submersible bridges are generally used in major district roads with low intensity of traffic when economy has to be achieved for shortage of funds and the choice is between low cost bridges or no bridges at all.

18.2.3.2 Submersible bridges are constructed with full linear waterway from bank to bank without any restriction of waterway. Both the banks shall be protected with pitching in line with the apron used for the protection of base raft. For submersible bridges, aprons for the U/S and D/S sides shall be 6 metres and 9 metres respectively upto the cut-off walls which shall be 2.0 metres for U/S side and 2.5 metres for D/S side. The details of submersible bridges are more or less the same as the vented causeway with rectangular openings except that openings in case of submersible bridges may be of larger dimensions. The guide/guard posts shall be provided as in causeways.

18.3 MOVABLE BRIDGES

18.3.1 Movable bridges may be defined as those bridges which carry land traffic for certain period and when required these are shifted or moved thereby causing interruption of land traffic but allowing other kind of traffic say river traffic. These bridges, therefore, may be termed temporary so far as their use is concerned but as regards their life, some of them may be permanent in nature.

18.3.2 There are various types of movable bridges such as (i) Swing bridge, (ii) Bascule bridge, (iii) Traverser bridge, (iv) Transporter bridge, (v) Cut-boat bridge, (vi) Lift bridge, (vii) Raft bridge, (viii) Boat bridge, (ix) Pontoon bridge.

18.3.3 Swing Bridge

18.3.3.1 Swing bridge is constructed over a channel through which steamers and ships ply. A central pier is constructed over the channel and a two span continuous truss bridge is built with end supports on two abutments on each bank and the central support over the central pier. The roadway is located at the top or bottom chord and vehicular traffic cross the channel from one bank to the other. When steamers or ships are to cross the bridge, the bridge is made to swing by 90 degrees i.e. from across the river to parallel to the river and thus the passage for the ships or the steamers is made.

18.3.3.2 Both the truss of the bridge are supported at the center over a common cross girder which rests on a central bearing or a pivot (Fig. 18.9a). This bearing is normally a phosphor bronze disc between two hardened steel discs. An alternative arrangement of support is by a circular girder resting on rollers, i.e., a turn-table (Fig. 18.9b). The stability of the trusses with central bearing, while swinging, is maintained by few balance wheels fixed to the truss or the floor system in such a way that they roll on a circular path over the piers.

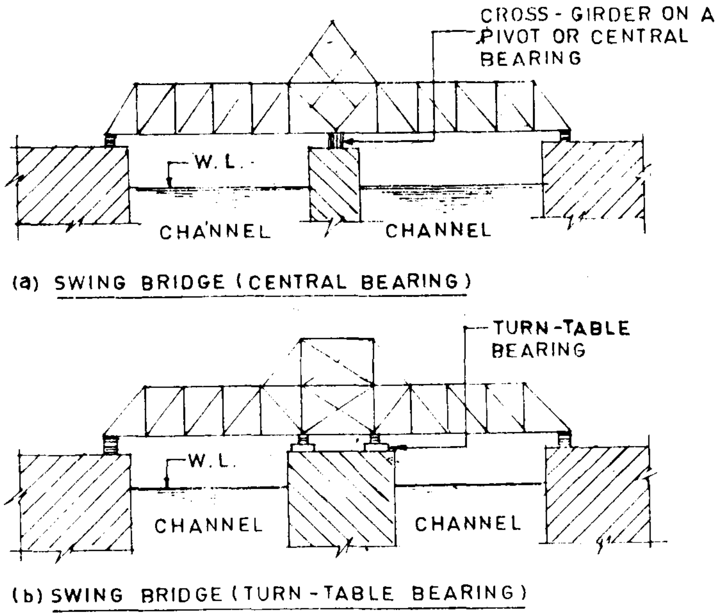


FIG. 18.9 SWING BRIDGE (Joglekar)[6]

18.3.4 Bascule Bridge

The bridges, when in use, remain in a horizontal position but having small free-board or the clearance over the water level, boats and other riverine transport can not cross the bridge unless these are lifted vertically. For small channels, single bascule bridge (Fig. 18.10a) is used but for large channels, double bascule bridge (Fig. 18.10b) is constructed since single bascule would be to heavy to operate. Formerly, a cable was used to lift the bascule but now a days the bascule is rotated by a motor through a rack and pinion arrangement. The rack is fixed to the bascule near or at the center of gravity of the bascule which is counter-weighted at the ends for the facility of easy lifting and keeping safely in the lifted position. There is a bascule chamber into which the counter weight of the bascule is lowered.

18.3.5 Traverser Bridge

Traverser bridge or draw bridge (Fig. 18.11) built over a small channel can be drawn towards the shore on one of the approaches when riverine traffic has to cross the bridge. The bridge traverses over rollers or wheels. When moves forward, the nose of the bridge acts like a cantilever for which the bridge truss has to be designed. A buffer is used to stop over-rolling of the rollers/wheels so as to prevent accidental fall of the bridge truss.

18.3.6 Transporter Bridge

This type of bridge (Fig. 18.12) consists of a travelling cage through which the traffic is transported. An overhead truss spanning the channel to be crossed by the traffic is supported by two towers erected on either bank of the channel. The travelling cage is supported over the truss through movable wheels and is transported bodily from one bank to the other when the cage carries the traffic as in ferry service. Therefore, transporter bridge works more as a ferry than a bridge for vehicular transport.

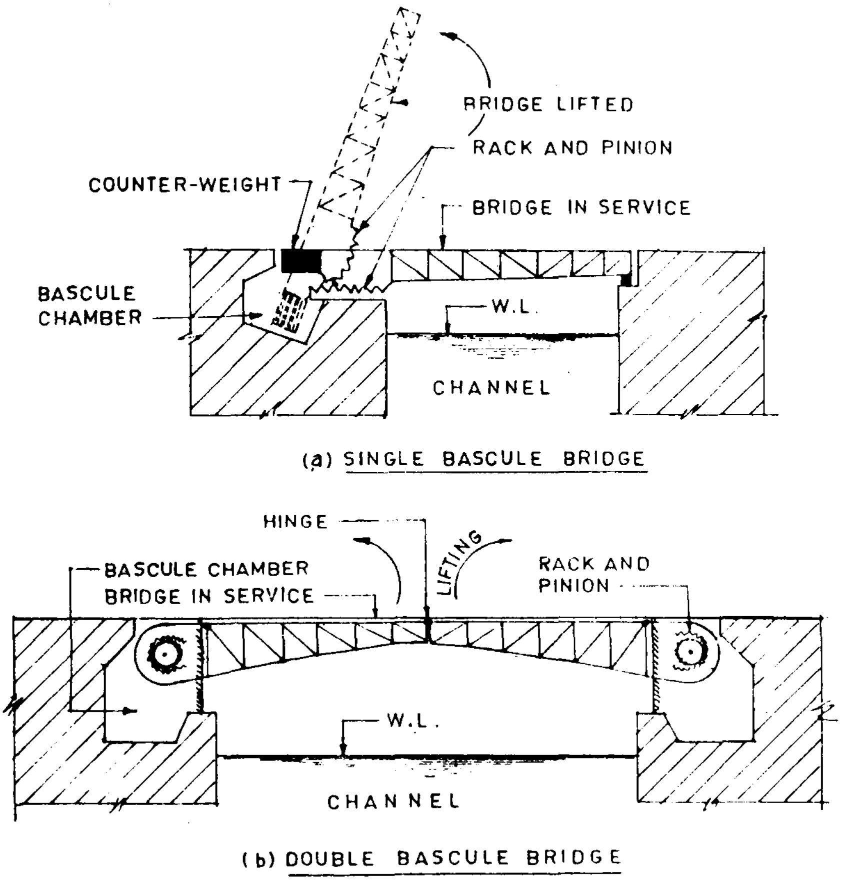


FIG. 18.10 BASCULE BRIDGES (Alagia)[4]

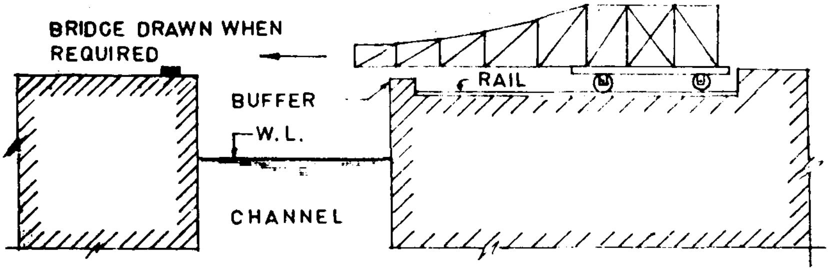


FIG. 18.11 TRAVERSER OR DRAW BRIDGE (Alexander)[5]

18.3.7 Cut-Boat Bridge

This is a boat bridge in which part of the bridge is cut and allowed to move downstream thus making passage for normal boat traffic. A portion of the bridge is constructed into an independent raft having cut on both ends and when required, one end of the raft is attached by strings to the main bridge while the other end is released to move downstream (Fig. 18.13).

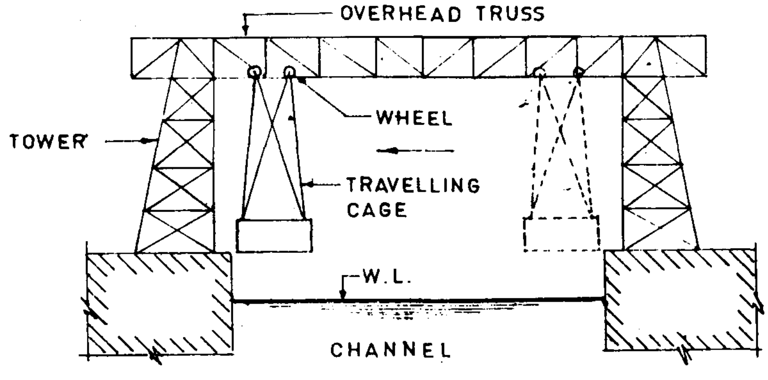


FIG. 18·12 TRANSPORTER BRIDGE (Alexander)[5]

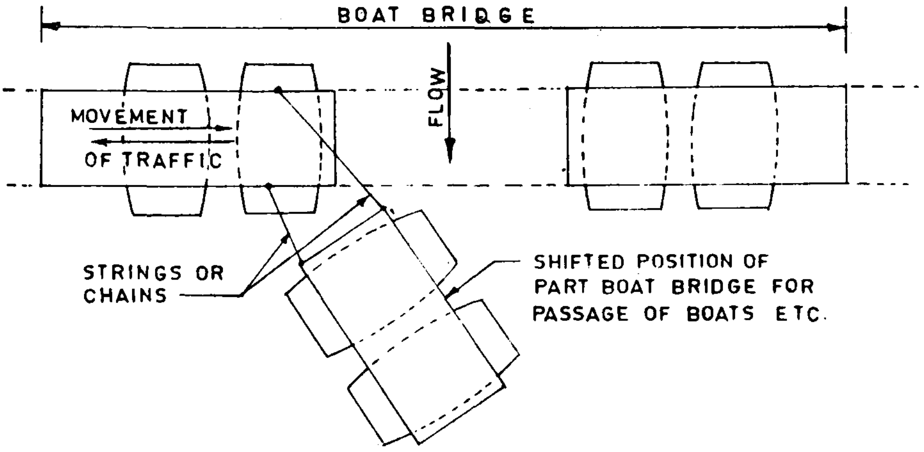


FIG. 18·13 CUT-BOAT BRIDGE (Joglekar)[6]

18.3.8 Lift Bridge

The lift bridge (Fig. 18.14) consists of a truss bridge deck which can be moved up and down by cables. Similar to the transporter bridge, two towers are erected on either bank of the channel and these are connected by an overhead truss for maintaining rigidity of the towers as well as for providing walkway for maintenance and support for cable ducts, water main etc. The truss bridge deck is lifted vertically by cables one end of which is fixed to the ends of the bridge and the other end passes over pulleys supported on the top of the tower and is ultimately connected to a counterweight. The bridge deck in lifted position allows boats and other rafts to cross the bridge underneath the deck. Lift bridges are economical both in construction and operation than the basucle bridges.

18.3.9 Raft Bridge

This is a floating bridge made of empty barrels closed at both ends and joined together in line along the flow thus forming the supports at some intervals (Fig. 18.15). The superstructure or the decking may be made of

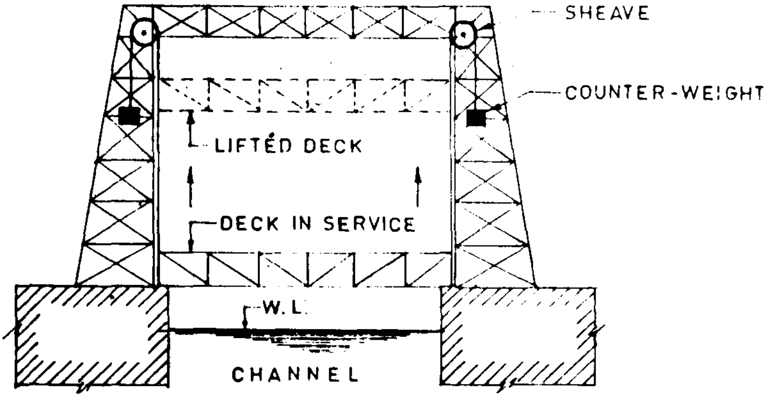


FIG. 18·14 LIFT BRIDGE (Alexander)[5]

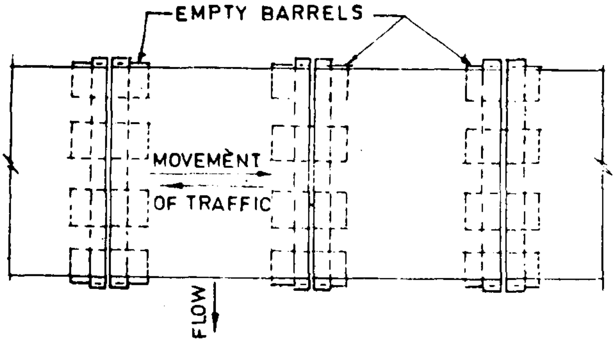


FIG. 18·15 RAFT BRIDGE (Plan) (Joglekar)[6]

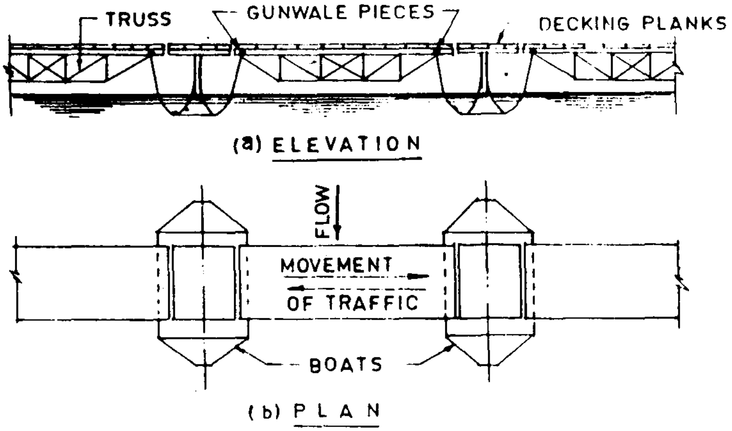


FIG. 18·16 BOAT BRIDGE (Alagia)[4]

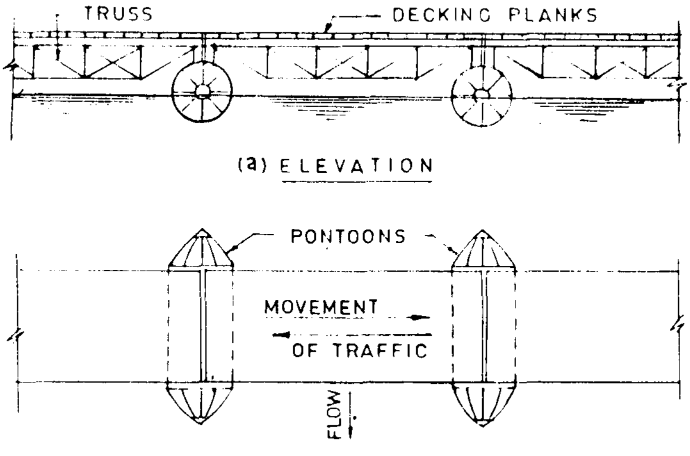


FIG. 18.17 PONTOON BRIDGE (Alagia)[4]

timber or steel and timber. Ramps are used at either end for the facility of movements of traffic from the shore to the bridge. Raft bridges are suitable for light vehicles.

18.3.10 Boat Bridge

This is also a floating bridge like a raft bridge but in this case large boats are used in place of empty drums as supports (Fig. 18.16). The axis of the boats lies along the flow and the boats are placed at regular intervals from one bank to the other. Since the boats are large size barges, they can carry more loads than the barrels and therefore, boat bridges have not only greater load carrying capacity but also have larger spans than the raft bridges. Therefore, the decking is made of steel girders or steel trusses over which timber decking is fixed. These bridges may be used by the civil authority as a temporary bridge in case of emergency or as a diversion bridge during repair or replacement of a major bridge.

18.3.11 Pontoon Bridge

Pontoon bridges are similar to boat bridges. In these bridges, larger size closed ended empty pontoons are used in place of boats. Since pontoon bridges have greater load carrying capacity and are quicker in construction, they are extensively used by the army during war time.

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CHAPTER 19

PIERS AND ABUTMENTS FOR BRIDGES

19.1 GENERAL

19.1.1 The piers and abutments are the supporting members through which the loads from the superstructure are transferred to the foundations. The piers support either intermediate spans coming from both ends or one shore span and one intermediate span while the abutments support only single spans or one end of shore spans. The abutments sometimes function as earth retaining structures in addition to transferring the vertical and horizontal loads from the superstructure to the foundation.

19.1.2 The layout of the piers and abutments either on land (viaduct, subway or overbridges) or on river should be such that streamline flow of traffic or water, as the case may be, is ensured.

19.2 TYPES OF PIERS

19.2.1 The materials used for the construction of bridge piers are (i) brick masonry, (ii) stone masonry, (iii) mass concrete, (iv) reinforced concrete and (v) prestressed concrete. Masonry or mass concrete piers are generally massive and therefore, offer more obstruction to linear waterway as well as increase the loads on foundations. Reinforced concrete or prestressed concrete piers, on the otherhand, have much reduced cross-sectional area and therefore, require much less foundation area in addition to offering less obstruction to waterway. Where brick and stone materials are costly, it is generally found economical to use R. C. or prestressed concrete piers.

19.2.2 The general shapes of various types of piers commonly used are illustrated in Fig. 19.1. From structural as well as from architectural considerations, the sides are usually made with a batter of 1 in 50 or 1 in 60. To ensure smooth and streamline flow of water, cut waters are provided in piers. The shape of these cutwaters may be semi-circular, arcs of circles, triangular etc. Piers are always provided with pier caps for transferring the loads to the pier shafts.

19.2.3 Masonry, mass concrete or R.C.C. solid shaft piers (Fig. 19.1a and 19.1b) are generally founded on open raft foundation where the possibility of scour is nil. Pile foundations or well foundations are also possible for such type of piers,

19.2.4 R.C.C. or prestressed concrete cellular piers (Fig. 19.1c) are suitable for major bridges where both the span and the depths are considerable and the self weight of the piers should be as minimum and the section modulus as maximum as possible. Intermediate diaphragms are provided horizontally to stiffen the vertical

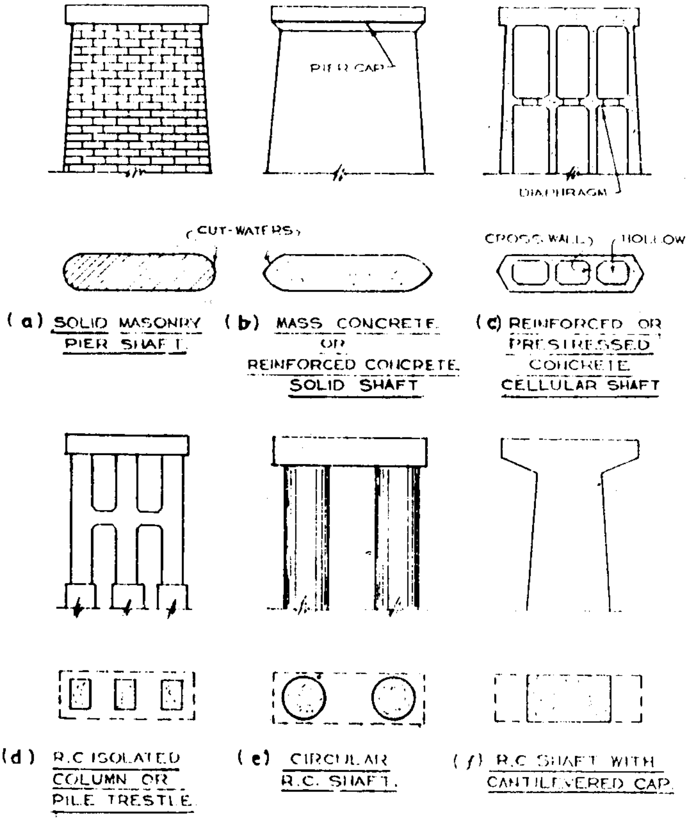

FIG.19·1 GENERAL SHAPE OF SOME TYPICAL PIERS

walls. Though the piers are covered with thick R.C. cap for even distribution of loads on the vertical walls, it is preferable to place the girders over the vertical cross-walls connecting the outer walls.

19.2.5 R.C.C. isolated square or circular columns (Fig. 19.1d & 19.1e) are preferred for highway bridges built on land. Isolated columns extended from R.C. piles or from well cap are also used for bridges built across rivers. The girders directly transfer the load to the columns. To make the columns rigid and to reduce the effective height, intermediate ties are provided.

The type of pier shown in Fig. 19.1f is most suitable for land bridges where the obstruction due to piers should be as minimum as possible so as to make room for normal traffic lanes on both sides of the piers by placing them centrally. The pier cap may be cantilevered out to support the superstructure.

Instead of making single rectangular pier as shown in Fig. 19.1f, single circular piers may also be used for land bridges. Single circular piers are most suitable for skew and curved bridges vide Art. 9.2.2.

19.3 DESIGN CONSIDERATIONS FOR PIERS

19.3.1 The masonry or mass concrete piers are designed with vertical loads and moments acting on the piers such that the resultant falls within or very near to the middle third line. By this limitation, it will be possible either to arrive at a no tension condition or to restrict the tension within the safe values.

19.3.2 In reinforced concrete piers, the concrete and the steel stresses due to vertical loads and moments are brought within the allowable limits. The section of the piers and the magnitude of the prestressing force in prestressed concrete piers are to be determined with a view to limit the maximum concrete stress within permissible limit. Generally, no tension is permitted in prestressed piers but slight tension not exceeding one-tenth of allowable compressive stress is often allowed when such tensile stresses are due to temporary loading conditions such as launching operation etc.

19.3.3 The loads and forces with which the piers are to be designed are :

i) Self weight of pier.
ii) Dead loads from adjacent spans and live load reactions either from one or from both spans whichever produces maximum effect.
iii) Buoyancy effect on the piers owing to pore pressure (usually taken as 15 percent)
iv) Horizontal force due to temperature effect and tractive or braking effect acting on the top of pier.
v) Horizontal force due to water-current acting on the pier at the center of gravity of the water pressure diagram.
vi) Horizontal force due to wind acting on the superstructure and the pier at the center of gravity of the respective wind pressure diagram.
vii) Centrifugal force acting on the pier when the bridge is on a curve.
viii) Horizontal force due to seismic effect on the superstructure as well as on the pier acting at the respective center of gravity.

19.3.4 The combination of the above loads and forces which may act together (Art. 5.1.2 of Chapter 5) should be such as to produce maximum effect.

19.4 REINFORCEMENT

19.4.1 In mass concrete piers, no reinforcement is required from structural considerations but nominal reinforcement at the rate of 5 Kg. for S240 grade steel and 3.5 Kg for S415 grade steel per square metre of the exposed surface is provided for temperature and shrinkage effect.

19.4.2 For reinforced concrete piers, the percentage of longitudinal reinforcement should neither be less than 0.8 nor more than 8 percent of the gross cross-sectional area. Where the cross-sectional area exceeds the concrete area required to support the vertical loads only, the percentage of reinforcement shall be calculated on the basis of the area required to resist the direct load and not on the actual area of the piers. In any case, the steel area shall not be less than 0.3 percent of the gross area. The lateral reinforcement or binders are provided in the piers at a spacing not less than 300 mm. The diameter of lateral reinforcement shall not be less than one-quarter the dia. of the largest longitudinal reinforcement nor less than 8mm. The spacing of the lateral reinforcement shall not exceed the least lateral dimension of the pier or twelve times the dia. of the smallest longitudinal bar whichever is lesser. Suitable link bars tieing the longitudinals and the laterals shall be provided at suitable intervals.

19.5 TYPES OF ABUTMENTS

19.5.1 Brick or stone masonry, mass concrete or reinforced concrete may be used in the construction of abutments. The types of abutments usually adopted in highway bridges are shown in Fig. 19.2. In open raft foundation, masonry or mass concrete abutments are commonly used as they provide the dead weight for the stability of such structure.

19.5.2 Pile trestle abutments are open type abutments in which the back-fill is not retained by the abutments but is allowed to spill through the spaces of the trestle and sloped embankment in front of the abutment is formed. The sloped embankment is protected by brick or boulder pitching from damage due to water-currents. This type of abutments have the advantage that no special structure is required to be made excepting raising of the piles and providing a pile cap at the top for supporting the superstructure. A dirt wall is required to be provided to prevent the dirt or the earth from the approaches spilling on the bearings. A screen wall as explained in Art. 19.5.4 is provided in all open type abutments.

19.5.3 Counterfort type abutments are closed type abutments having some columns or counterforts connected by a face slab in front. The spacing of the counterforts are generally 2.5 to 3 metres. The stability of the abutments is maintained by the self weight and the weight of the back-fill materials in between the counterforts and above the foundation raft. Open raft or pile or well foundation is suitable for this sort of abutments.

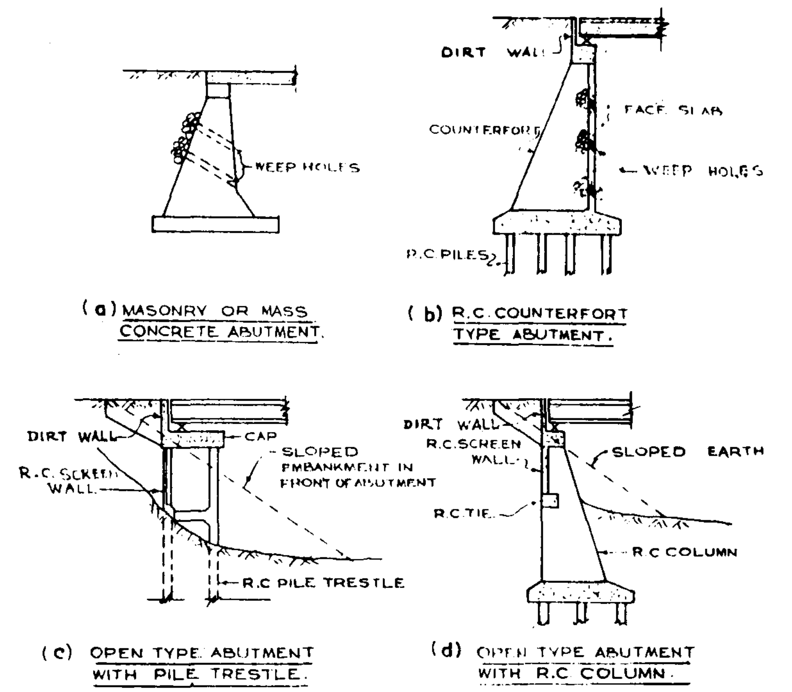

FIG.19·2 VARIOUS TYPES OF ABUTMENTS

19.5.4 Open type abutments with R.C. columns (Fig. 19.2d) are preferred where the height of formation is very high. In order to relieve the abutments from excessive earth-pressure, the earth is allowed to spill in front as in pile trestle abutments. The spacing of the columns is also more or less similar to counterfort spacings i.e., 2.5 to 3.0 metres. A screen wall about 1.5 to 2.5 metre deep is required to be provided connecting the columns and hanging from the capping beam. The function of this screen wall is to prevent movement of earth from the top area just behind the abutment due to surcharge, vibration etc. Foundations required for such type of abutments are either pile or well foundations. Raft foundation may be possible if it rests on rock.

The advantage of open type abutment is that no wing or return walls are necessary but the disadvantage is that some waterway is restricted by the sloped embankment in front of the abutments. On the otherhand, closed type abutments require wing walls or return walls for protection of earth but these abutments ensure more waterway than the open type abutments do.

19.6 DESIGN CONSIDERATIONS FOR ABUTMENTS

19.6.1 The abutments are subjected to the following loads and forces :

i) Self weight of abutments including the weight of the back-fill materials over the abutments.

ii) Dead and live load from superstructure — minimum live load for checking tension and maximum live load for checking maximum compression.

iii) Temperature and tractive or braking effect.

iv) Horizontal force due to wind on superstructure.

v) Centrifugal force if the bridge is on a curve.

vi) Active earth pressure at the back including live load surcharge effect. All abutments shall be designed for a live load surcharge equivalent to 1.2 metre height of earthfull.

vii) Seismic force.

19.6.2 The capping slab for open or counterfort type abutment is to be designed for both vertical and horizontal loads. Usually, the abutment caps are subject to torsional stresses and adequate torsional reinforcement is required to be provided. The earth pressure on the columns for open type abutments should take into account the arch action of the soil mass exerting pressure. To cater for this effect, an increase of the earth pressure to the extent of 100 percent on such abutment columns is normally assumed.

19.6.3 It is very important to check the stability of the abutments as a whole in respect of sliding and overturning when these abutments are founded on open rafts. The tendency of the abutments to sliding due to the horizontal force is resisted by μV, where μ is the coefficient of friction between the soil and the base of foundation and V is the total vertical load on the foundation. Adequate factor of safety against failure should be allowed. Let H be the total horizontal sliding force and V be the total vertical load. For stability,

$$\frac{\mu V}{H} \not< 2.0$$

The value of μ is taken as equal to $\tan \phi = \tan 20^\circ$. Similarly, there must be sufficient margin of safety against overturning of the abutments as a whole about the toe. This may be given by,

$$\frac{\Sigma M_s}{\Sigma M_o} \not< 1.5$$

Where ΣM_S = Sum of all the stabilizing moments about toe

ΣM_O = Sum of all the overturning moments about toe

19.6.4 In closed type high abutments, the total earth pressure on the walls are comparatively on the high side and therefore, only the base friction μV may not be capable of resisting the sliding of the abutments. In such cases, shear key as shown in Fig. 19.3 is used to increase the resistance to sliding. The passive earth pressure in front of the shear key is taken advantage of for the purpose. Some authority recommends that passive resistance may be calculated taking the modulus of horizontal subgrade reaction as 0.7 times the vertical one. The passive resistance offered by the earth in front of the walls may also be taken advantage of if it is well compacted virgin soil and no possibility of scouring away of the earth in front is apprehended.

19.6.5 The theory of earth pressure and the design of the gravity type or counterfort type walls may be found in any book on Theory of Structures and therefore, is not discussed here. The open type abutments may be designed in the manner indicated below.

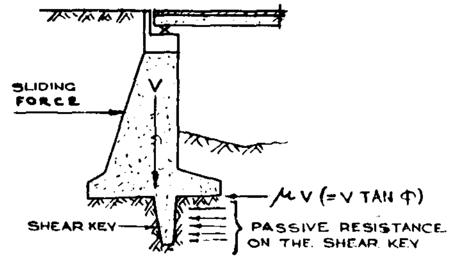

FIG.19·3 ADDITIONAL PRECAUTION AGAINST SLIDING FAILURE

The screen walls are continuous over the columns and fixed at top with the cap. If tie is provided at bottom, the wall may be assumed as simply supported but sometimes the bottom is kept free without any support whatsoever. In that case, the bottom is assumed as free cantilever. The earth pressure on the screen wall is distributed over it in both directions taking into consideration the support conditions. The columns are designed with vertical loads and moments caused by earth pressure and other forces similar to members subjected to direct thrust and bending. The foundation raft is a continuous slab over the column supports with soil pressure below and may be designed as such.

19.6.6 Seismic Effect of the Back Fill on the Abutments

During seismic disturbances, the back fill material behind the abutments also vibrate and therefore, exerts increased earth pressure which may be computed as detailed below.

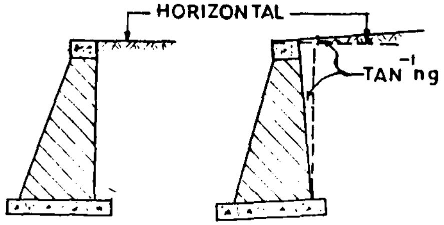

FIG.19·4 ASSUMED ANGULAR DISPLACEMENT OF WALL DUE TO SEISMIC EFFECT (Teng)[5]

Due to the action of the earthquake having seismic coefficient, n_g, both the abutment wall and the back-fill are assumed an imaginary displacement of $\tan^{-1} n_g$ (vide Fig. 19.4) and the unit weight of the back-fill is increased by multiplying it with a factor of $\sqrt{1 + n_g^2}$. The earth pressure calculated by the usual theory with the above modifications gives the increased effect due to seismic disturbance in the backfill materials. In addition to the increased earth pressure, the seismic effect on the abutment itself should be considered in the normal way.

ILLUSTRATIVE EXAMPLE 19.1

Calculate by Coulomb's theory the horizontal component of normal earth pressure and that with earthquake effect on the back-fill. Find the percentage increase in the normal earth pressure when seismic effect on the back-fill is considered.

Given :
$\alpha = 90°$
$\phi = 30°$
$\delta = \frac{\varphi}{2} = 15°$
$\beta = 0$
$\gamma = 1800\ \boldsymbol{Kg/m^3}$
$n_g = 0.05$ ***and*** 0.10

Normal earth pressure

As per Coulomb's theory,

$$P_A = \frac{1}{2}\gamma H^2 \times K_A \tag{19.1}$$

Where,

$$K_A = \frac{\sin^2(\alpha+\varphi)}{\sin^2\alpha \sin(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\varphi+\delta)\sin(\varphi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2}$$

$$= \frac{\sin^2(90+30)}{\sin^2 90 \sin(90-15)\left[1+\sqrt{\frac{\sin(30+15)\sin(30-0)}{\sin(90-15)\sin(90+0)}}\right]^2} = 0.30$$

$$\therefore\ P_A = \frac{1}{2}\times 1800 \times 0.30\ H^2 = 270\ H^2$$

$\therefore$ Horizontal component, $P_{AH} = 270\ H^2 \cos\delta = 270\ H^2 \cos 15° = 261\ H^2$

Earth Pressure with Seismic Effect

(a) *When seismic coefficient, $n_g = 0.05$*

$\therefore\ \tan^{-1} ng = \tan^{-1} 0.05 = 2° - 50'$ $\quad\therefore\ \gamma' = \gamma\sqrt{1+n_g^2} = 1800\sqrt{1+(0.05)^2} = 1802\ Kg/m^3$

$$\therefore\ \beta = 2° - 50' \text{ and } \alpha = 90° - (2° - 50') = 87° - 10'$$

From equation 19.1,

$$K_A = \frac{\sin^2(87°10' + 30°)}{\sin^2 87°10' \times \sin(87°10' - 15°)\left[1+\frac{\sin(30°+15°)\sin(30°-2°50')}{\sin(87°10'-15°)\sin(87°10'+2°50')}\right]^2} = 0.33$$

$P_A = \frac{1}{2}\gamma' H^2 K_A = \frac{1}{2}\times 1802 \times 0.33\ H^2 = 297\ H^2$; $\quad P_{AH} = P_A \cos(\delta+\beta) = 297\ H^2 (15° + 2°50') = 283\ H^2$

$\therefore$ Percentage increase of horizontal earth pressure due to seismic effect having seismic coefficient of 0.05

$$= \frac{(283\ H^2 - 261\ H^2)}{261\ H^2}\times 100 = 8.4$$

(b) *When seismic coefficient, $n_g = 0.10$*

$$\therefore \quad \tan^{-1} n_g = \tan^{-1} 0.10 = 5°43'$$

$$\gamma' = \gamma\sqrt{1 + ng^2} = 1800 \times \sqrt{1 + (0.10)^2} = 1809 \text{ Kg/m}^3$$

$$\beta = 5°43' \text{ and } \alpha = 90° - 5°43' = 84°17'$$

Putting these values in equation 19.1, the value of K_A comes to 0.367.

$$\therefore \quad P_A = \frac{1}{2}\gamma' H^2 K_A = \frac{1}{2} \times 1809\ H^2 \times 0.367 = 332\ H^2$$

$$P_{AH} = P_A \cos(\delta + \beta) = 332\ H^2 \cos(15° + 5°17') = 311\ H^2$$

$\therefore$ Percentage increase of horizontal earth pressure due to seismic effect having seismic coefficient of $0.10 = \frac{(311 - 261)}{261} \times 100 = 19.16$

Therefore, it is noted that due to seismic effect, the horizontal earth pressure increases by nearly 10 percent and 20 percent when the seismic coefficients are 0.05 and 0.10 respectively.

19.6.7 Live Load Surcharge

The abutments shall be designed for a live load surcharge equivalent to 1.2 metre height of earth fill.

19.6.8 Weep Holes

In closed type abutments, adequate number of weep holes (Fig. 19.2) shall be provided to drain out the water accumulated at the back of the abutments otherwise additional horizontal pressure will be exerted by the accumulated water on the abutments. The weep holes shall be made at a dip on the outer side for the facility of easy drainage. The back of the weep holes shall be properly packed and protected with filter materials of varying sizes, the larger size being in contact with the wall so that neither the back fill materials nor the filter materials can get out through the weep holes. The size of the weep holes may be 150 mm deep and 75 mm wide and the spacing shall not exceed one metre in both horizontal and vertical directions.

19.6.9 Back-fill materials

The back-fill shall be of granular materials as far as possible. Sandy soils or sandy silts may also be used if granular materials are not available. The optimum moisture content of such granular materials shall be between 7 to 10. The filter material behind the weep holes as mentioned in Art. 19.6.8, if used at the entire back area of the abutments, will help quicker drainage of the accumulated water and as such the latter provision is better than the former one viz. use of localized filter materials just behind the weep holes.

19.7 REFERENCES

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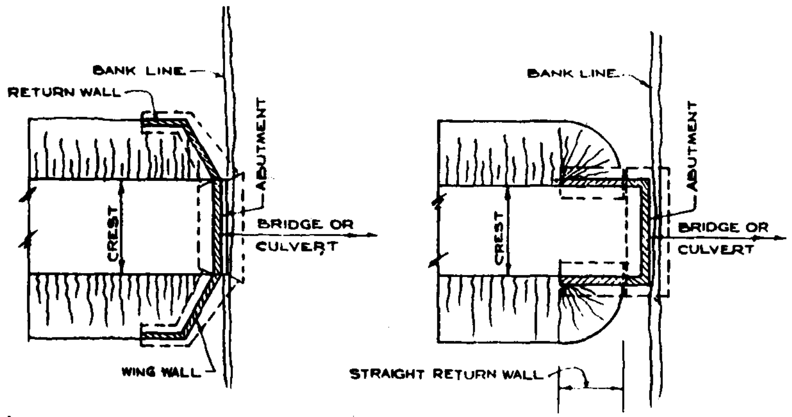
CHAPTER *20*

WING WALLS AND RETURN WALLS

20.1 GENERAL

20.1.1 In the previous chapter, the details of the bridge abutments have been discussed. Closed type abutments are used where the spilling of the earth in front of the abutments is to be prevented by retaining the earth and therefore, such sort of abutments functions as retaining walls in addition to acting as load bearing walls. In bridges provided with closed type abutments, the sides are also to be protected by walls so as to prevent the spilling of the earth. These walls when placed at an angle with the road embankment in the form of *"wings"* are known as *"wing walls"* whereas they are termed as *"return walls"* when placed parallel to the embankment (Fig. 20.1). Retaining wall is the general term of the wall which retains earth and as such the wing walls and the return walls are also retaining walls.



(a) WING & RETURN WALL.
(b) STRAIGHT RETURN WALL.

FIG.20·1 WING AND RETURN WALLS

20.2 TYPES OF WALLS

20.2.1 Retaining walls may be built of brick or stone masonry, cement concrete or reinforced cement concrete. The following types of retaining walls are generally used :

i) Gravity or semi-gravity wall.
ii) Cantilever wall.
iii) Counterfort wall.
iv) Buttress wall.
v) Tied-back walls.

20.2.2 Fig. 20.2 illustrates various types of retaining walls. Gravity walls require massive sections and therefore, masonry or cement concrete is used in such walls. Reinforced cement concrete thin sections are used in the construction of cantilever, counterfort or buttress walls. Gravity walls may be suitable upto height of 6 metres. Cantilever walls are generally adopted up to a nominal height of 6 metres. When the nominal height exceeds 6 metres, counterfort or buttress type walls are used. Tied-back walls may be used for high walls. These walls are specially suitable in cases where walls on both the sides are to be provided.

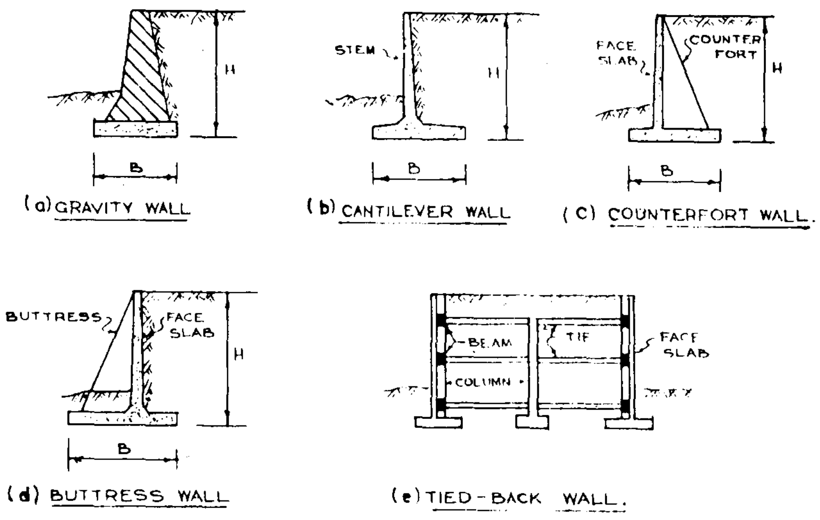


FIG.20·2 VARIOUS TYPES OF RETAINING WALLS

20.3 PROPORTIONING OF MEMBERS

20.3.1 In gravity type walls, the base width is kept as 2/3 the overall height of the wall. Usually a batter of 1 in 20 is provided in the front face where a haunch of one horizontal to two vertical for a depth of about $\frac{1}{4}$ height near the base is also provided from stability consideration.

20.3.2 The base width of the cantilever, counterfort or buttress walls varies from $\frac{1}{2}$ to $\frac{1}{3}$ the height. The projection of the toe from the face of the wall is $\frac{1}{3}$ the base width for cantilever or counterfort walls. The stem thickness of the cantilever walls is $\frac{1}{12}$ the height and the thickness of the base raft is $\frac{1}{8}$ to $\frac{1}{12}$ the height. The spacing of the counterforts or the buttresses or the columns of the tied-back walls should be between 2.5 to 3.5 metres. The width of the counterforts or buttresses is generally 450 to 600 mm. Tie-beams of section 500 × 200 mm

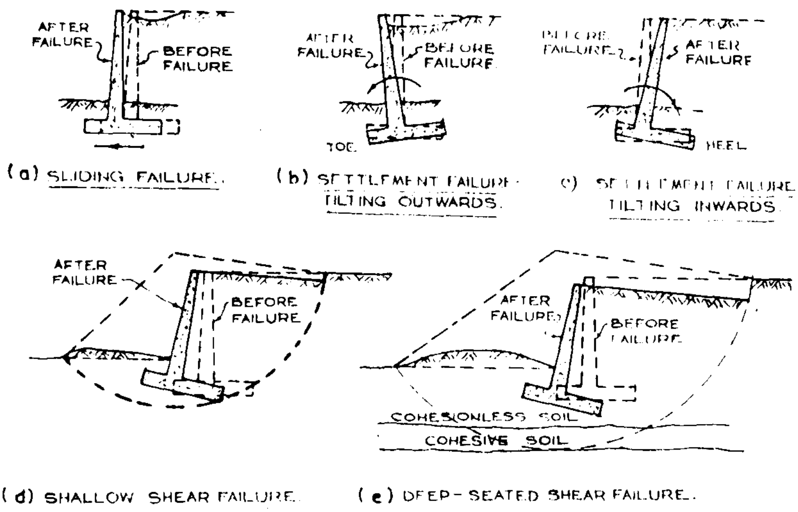


FIG.20·3 FAILURE OF WALLS DUE TO VARIOUS CAUSES (Huntington)[4]

to 700 × 250 mm are normally found adequate for the tied-back walls. The top of the tie walls is made of inverted V-shape to minimize direct earth load including live load surcharge (Refer Fig. 20.4).

20.4 DESIGN CONSIDERATIONS

20.4.1 Similar to the abutments, the stability of the walls against sliding or overturning is very important in addition to the safety of the walls in respect of safe foundation pressure and therefore, must be checked in the manner already explained in the previous chapter. Retaining walls are more susceptible to failure by overturning than the abutments for the reason that there is no vertical superimposed load on the walls as in abutments except the self weight and the weight of earth coming over them. Failure of the walls may also take place due to the following reasons :

i) Sliding failure (Fig. 20.2a)

ii) Settlement failure (Fig. 20.3b & c)

iii) Shallow shear failure (Fig. 20.3d)

iv) Deep-seated shear failure (Fig. 20.3e)

20.4.2 Sliding failure may occur when the sliding resistance at the base or the shearing resistance of the soil under the base is small compared to the horizontal thrust exerted on the wall.

20.4.3 Settlement failure is caused due to the excessive settlement of the foundation soil. The wall may tilt outwards when the toe pressure is more than the allowable foundation pressure. On the otherhand, the inward tilting of the wall takes place if the soil under the heel is of poor bearing capacity.

20.4.4 Shallow shear failure occurs when the wall rests on a soil having very poor shear strength (Fig. 20.3d). When the wall is founded on a cohesionless soil with good shearing resistance but the soil underneath the cohesionless soil is cohesive with less shearing resistance, shallow shear failure cannot take place but the wall may move along with the cohesionless soil underneath the wall at the plane of weakness resulting in deep-seated shear failure (Fig. 20.3e).

20.4.5 After checking the stability of the walls, the foundation pressure coming on the soil both at the toe and heel with worst condition of loading may be investigated and compared with the permissible value. If this is satisfactory, then the adequacy of the structural components such as foundation rafts, walls, counterforts, buttresses, columns, ties etc. must be examined.

20.4.6 The vertical stem or the wall of both the gravity and the cantilever retaining walls acts as a cantilever in the vertical plane under the action of the horizontal thrust exerted by the earth pressure. In the counterfort or the buttress type, the face slab spans horizontally between the counterforts or the buttresses as the case may be like a continuous beam causing bending of the face slab in the horizontal plane. The thrust from the face slab is transferred on to the counterforts or the buttresses which again behave like cantilevers similar to cantilever walls.

20.4.7 The tied-back walls are somewhat different in action from other walls. The face wall is supported on four sides by the vertical columns and the horizontal beams and as such the thrust exerted by the active earth pressure on the face wall is ultimately transferred to the node points, i.e. to the junction of beams and columns and the thrust is resisted by the pull in the ties. The face wall is designed as a slab supported on four sides. The horizontal beams are designed with the triangular or trapezoidal load from the face wall. For example, in Fig. 20.4, horizontal beam B_3 will have earth pressive load from face wall such as top trapezium "defg" and bottom trapezium "hklm".

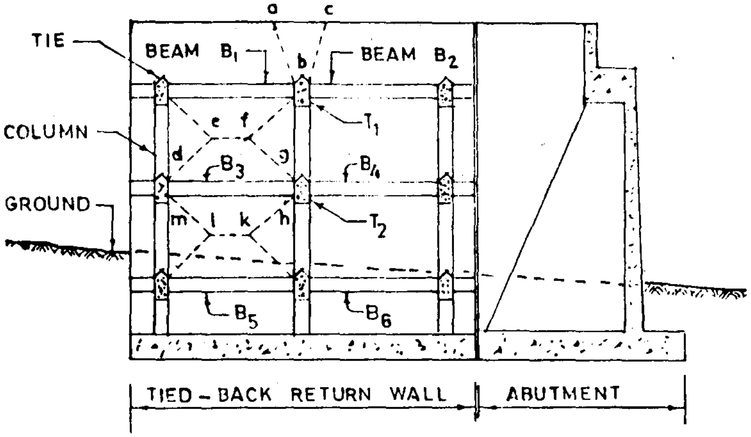


FIG. 20·4 TIED-BACK RETURN WALL : LOAD ON TIE (Chettoe et al)[3]

The load on the ties due to self weight, earth load etc. over them are transferred to the columns and therefore, the columns are to be designed with direct load from ties and moment caused by the load from face wall directly on the columns and the moment transferred from horizontal beams. The ties are designed with self weight, the earth load and the live load surcharge over them. It is believed that when the tie beam deflects, not only the weight of the earth directly over it comes on it but also some more earth as shown in Fig. 20.4 transfers the load over the tie due to arch action. For example, the weight of earth for the portion "abc" comes on top tie T_1. The live load surcharge effect is, however, assumed on the top tie only and neglected for the remaining ties. In calculating the live load surcharge on the tie-beam, the load coming on the portion "abc" is taken as the load per running metre of the tie-beam but this load should be judiciously taken. The Author suggests that the actual load (earth load and L.L. surcharge) directly coming on the tie beam T_1 may be increased by 100 percent to account for the arching action. The tension in the tie shall also be considered in the design.

20.4.8 Live Load Surcharge

All wing/return walls provided for full height of approaches shall be designed to withstand a live load surcharge equivalent to 0.6 metre height of earthfill.

20.4.9 Weep Holes

All wing/return walls shall be provided with adequate number of weep holes in the manner as described in Art. 19.6.8.

20.4.10 Back-fill Materials

Back-fill materials shall be as specified in case of abutments (Refer Art. 19.6.9).

20.5 REFERENCES

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CHAPTER **21**

SHALLOW AND DEEP FOUNDATIONS FOR BRIDGES

21.1 GENERAL

21.1.1 The function of a bridge foundation is to distribute the loads coming from the superstructure through the piers or abutments over the foundation materials so that these are able to take the loads without failure either due to excessive shear or excessive settlement of the foundation materials. In the former case, the soil or rock over which the foundation rests is sheared off from the surrounding medium and the structure fails as a whole due to shear failure. On the other hand, excessive settlement of the soil induces undue stresses in the structure itself supported over the foundation and though the structure does not show any sign of total failure due to failure of the medium, the undue stresses caused by the settlement produce cracks in the structure and the structure thereby is damaged. The former is known as the *"Bearing capacity"* failure whereas the latter one is termed as *"Settlement"* failure.

21.1.2 The factor of safety against a bearing capacity failure is normally kept as 3.0 over the ultimate bearing capacity but if the foundations are designed for the extreme loading conditions, a factor of safety of 2.0 may sometimes be allowed.

21.1.3 Slight settlement is allowed in many structures but this allowable settlement depends on the type of structures. In freely supported or balanced cantilever type superstructures, allowable settlement may be more than that in continuous or rigid frame or arch bridges. Again, uniform settlement may not be harmful if it occurs simultaneously throughout the structure. Differential settlement, even if lesser in magnitude, produces more serious effects than the uniform settlement does.

21.2 CLASSIFICATION

The foundations may be broadly classified as :

(A) Shallow foundation, and

(B) Deep foundation

The latter group may again be divided into :

(1) Pile foundation.

(2) Well foundation.

Under pile foundation, the following types may be mentioned :

(a) Timber piles
(b) Cast-in-situ concrete piles
(c) R. C. precast piles
(d) Tubular steel piles
(e) Screw piles

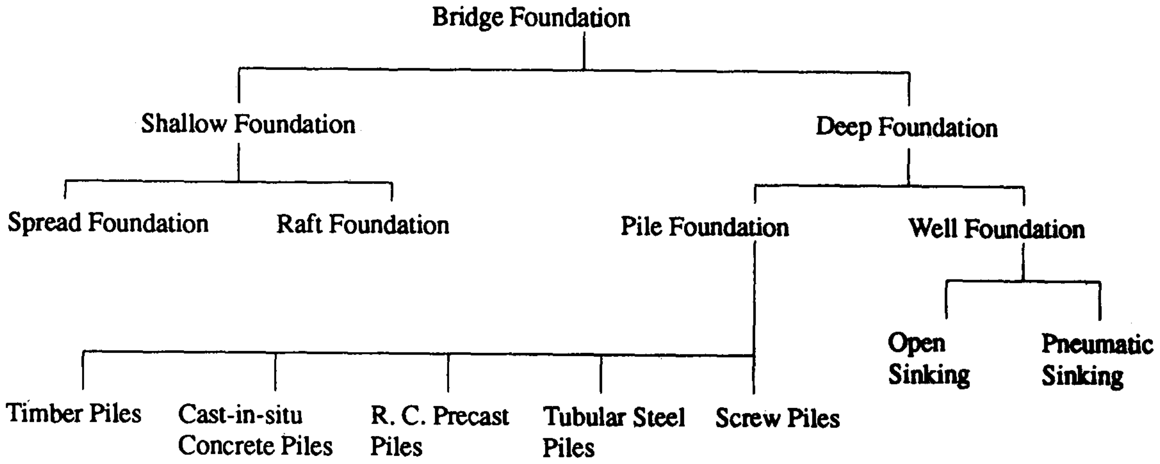
Well foundation is further subdivided depending upon the method of sinking, viz. :

(a) Open well sinking
(b) Pneumatic well sinking

In both the well types, sinking may be done by

(i) Making island
(ii) Making cofferdam
(iii) Launching caisson

The key diagram given below shows the classification of foundations at a glance.



21.3 SHALLOW FOUNDATIONS

21.3.1 Shallow foundations are normally defined as those whose depths are less than their widths. The foundations for masonry, mass concrete or R. C. Piers and abutments of lesser heights supporting comparatively smaller spans and having no possibility of any scour are normally made shallow. In cases, where the foundation materials are such that safe bearing capacity is very low within the shallow depth, this sort of foundations, though otherwise suitable, may not be advisable and deep foundation may be resorted to.

21.3.2 Design of the Footing

21.3.2.1 If the foundation footing is subjected to direct load only, the foundation pressure may be obtained by dividing the load with the area of the raft. If, however, it is subjected to moment in addition to the direct load, the maximum and minimum foundation pressures are calculated as below :

$$f_{max.} = \frac{P}{A} + \frac{M}{Z} = \frac{P}{A} + \frac{P \cdot e}{Z} \tag{21.1}$$

$$f_{min.} = \frac{P}{A} - \frac{M}{Z} = \frac{P}{A} - \frac{P \cdot e}{Z} \tag{21.2}$$

Where P = direct load
A = area of foundation footing
M = moment acting on the footing
e = eccentricity of the resultant from centre line of footing
Z = Section modulus of the footing.

21.3.2.2 For rectangular footing, no tension in the foundation will develop if the resultant of the combined effect of direct load and moment remains within the middle third of the base. If the resultant falls just on the middle third line, the maximum foundation pressure in that case is equal to twice the direct pressure and the minimum equal to zero. When the resultant exceeds the middle third line, tension develops and therefore, the entire foundation area does not remain effective in sustaining the load coming over it. Equation (21.1) does no longer remain valid in estimating the maximum foundation pressure which may be done as explained below :

When tension develops in part of the raft, part of the foundation width remains ineffective and the total load is taken by the remaining width.

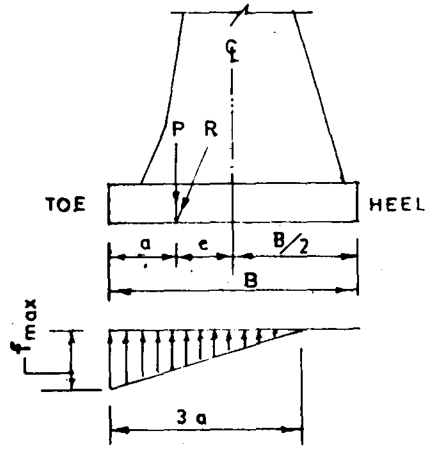


FIG. 21·1 FOUNDATION PRESSURE WITH RESULTANT OUTSIDE MIDDLE THIRD (Modified base area)

The point of application of the resultant is at a distance of "a" from the toe. In order to develop no tension condition on the modified effective width, the resultant must pass through the middle third line and therefore, the effective width must be equal to "3a" to satisfy the middle third condition. The total foundation pressure per metre length of footing must be equal to the vertical load, P, i.e., the load coming on the footing per metre length.

Assuming one metre length of wall

$$\frac{1}{2} \times f_{max.} \times 3a = P \quad \therefore \quad f_{max.} = \frac{2}{3}\ \frac{P}{a} \tag{21.3}$$

21.3.2.3 Generally, in foundations resting on soil, no tension is permitted. When the foundation rests on rock, tension may be allowed provided the maximum foundation pressure is calculated on the basis of the actual area carrying the load as outlined by equation (21.3). The foundation raft in this case needs adequate anchorage with the foundation rock by dowel bars.

21.3.2.4 The stability of the structure in respect of sliding and overturning should be checked as mentioned in Art. 19.6.3 in connection with the design considerations for abutments. The adequacy of the footing may be checked in respect of moments and shears considering the soil reaction at the base as determined by the method stated previously and the weight of the soil over the footing if the latter consideration governs the design. The reinforcement may be provided accordingly if it is of reinforced concrete.

ILLUSTRATIVE EXAMPLE 21.1

Design the foundation raft of a bridge pier with a direct load of 270 tonnes and a moment of 110 tonnes — metre about longer axis at the base of pier. The foundation raft rests on rock having a safe bearing pressure of 65 tonnes per square metre. Length of the raft is 7.5 m.

Solution

Assume size of raft = 7.5 m × 1.7 m $\therefore$ Area of raft = 7.5 × 1.7 = 12.75 m^2

Section modulus of base, $Z = \frac{7.5 \times (1.7)^2}{6} = 3.61\ m^3$

Total direct load at base of raft including self weight of raft @ 10 percent = 270 + 27 = 297 tonnes.

Assuming 10 percent increase of moment at the foundation base, total moment = 110 + 11 = 121 tm.

$$\therefore\ f_{max} = \frac{P}{A} + \frac{M}{Z} = \frac{297}{12.75} + \frac{121}{3.61} = 23.29 + 33.52 = 56.81\ \text{t/m}^2$$

Allowable foundation pressure = 65.0 t/m^2, Hence safe.

$$f_{min} = \frac{P}{A} - \frac{M}{Z} = 23.29 - 33.52 = (-)\ 10.23\ \text{t/m}^2\ \text{(tension)}$$

Since the foundation raft rests on rock, tension may be permitted provided the raft is adequately anchored with the foundation rock with anchor bars and the maximum foundation pressure is calculated on the basis of effective area supporting the load as stated in Art. 21.3.2.2.

Foundation pressure on the modified area

Eccentricity of the resultant, $e = \frac{M}{P} = \frac{121}{297} = 0.41$ m,

From Fig. 21.1, $a = \frac{B}{2} - e = 0.85 - 0.41 = 0.44$ m

Effective base width supporting the load = 3a = 3 × 0.44 = 1.32 m

From equation 21.3, $f_{max} = \frac{2}{3} \times \frac{P}{a} = \frac{2}{3} \times \frac{297}{0.44} = 60.0\ \text{t/m}^2$

Allowable foundation pressure = 65.0 t/m^2, hence safe.

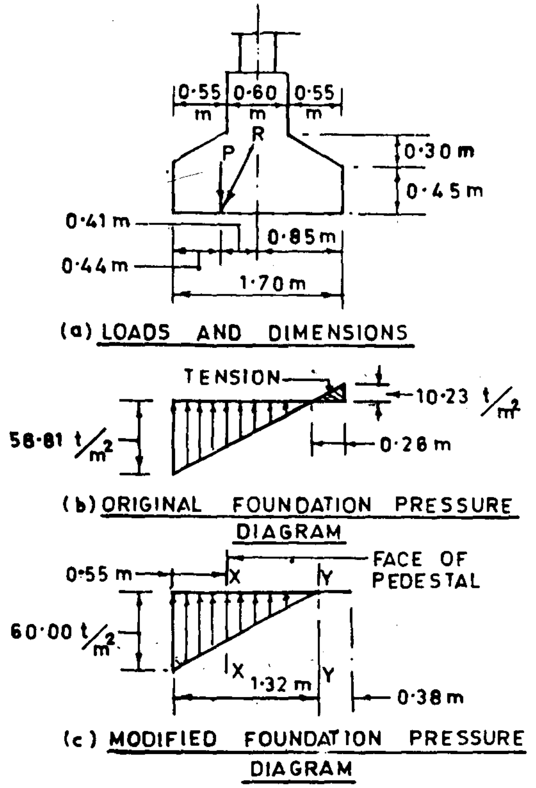


FIG. 21·2 FOUNDATION PRESSURE UNDER RAFT (Example 21·1)

Design of Bottom Steel

Weight of raft per m² = $\frac{27}{12.75}$ = 2.12 t/m²

Suppose, 1.2 m height of earth acts over the foundation raft.

Load due to this = $1.2 \times 1.8 = 2.16$ t/m².

Total downward load on the raft = 2.12 + 2.16 = 4.28 t/m²

Foundation pressure at X-X = $60.0 \times \frac{(1.32 - 0.55)}{1.32} = 35.0$ t/m²

Moment at X-X, i.e., at the pier face due to upward reaction per metre width =

$$35.0 \times \frac{(0.55)^2}{2} + \frac{1}{2}(60.0 - 35.0) \times 0.55 \times \frac{2}{3} \times 0.55 = 5.29 + 5.04 = 10.33 \text{ tm}$$

Less for downward moment due to self weight of raft and weight of earth over it = $4.28 \times \frac{(0.55)^2}{2} = 0.65$ tm

$\therefore$ Nett moment = 10.33 – 0.65 = 9.68 tm = 94,900 Nm

Referring to Illustrative Example 7.1 : $\sigma_c = 6.7$ MPa, $\sigma_s = 200$ MPa & $R = 0.95$

$$\therefore \quad d = \sqrt{\frac{94{,}900 \times 10^3}{0.95 \times 10^3}} = 316 \; mm$$

Effective depth available with 75 mm clear cover for foundation raft and 10 mm. for half dia of bar = 750 − 85 = 665 mm. Using HYSD bars,

$$A_s = \frac{94{,}900 \times 10^3}{200 \times 0.894 \times 665} = 800 \text{ mm}^2$$

Adopt 12 Φ @ 125 ($A_s = 904 \text{ mm}^2$)

Distribution steel = 30% of main steel = 0.3 × 800 = 240 mm², Use 8 Φ @ 175 ($A_s = 285 \text{ mm}^2$)

Top Steel

Due to tension in the foundation on either side, there will not be any contact between the raft and the soil underneath and therefore, the self weight of raft and the weight of earth over it will produce downward (i.e., hogging) moment.

Hogging moment at Y-Y (Fig. 21.2-C) = $4.28 \times \frac{(0.38)^2}{2} = 0.31$ tm = 3030 Nm

Effective depth at Y-Y = 657 − 80 = 577 mm $\quad \therefore \quad As = \frac{3030 \times 10^3}{200 \times 0.894 \times 577} = 29 \text{ mm}^2$

Every third bottom bar may be taken upwards ($A_s = \frac{1}{3} \times 904 = 300 \text{ mm}^2$)

Shear

Shear stress will be much within allowable limits.

Anchor bars

Total tension on the foundation raft (Fig. 21.2-b) = $\frac{1}{2} \times 10.23 \times 0.26 \times 7.5 = 9.97$ tonnes = 97,700 N.

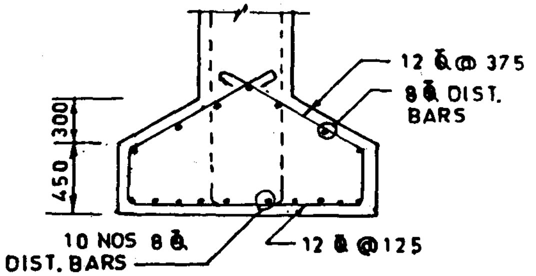


FIG. 21·3 DETAILS OF REINFORCEMENT FOR FOUNDATION RAFT (Example 21·1)

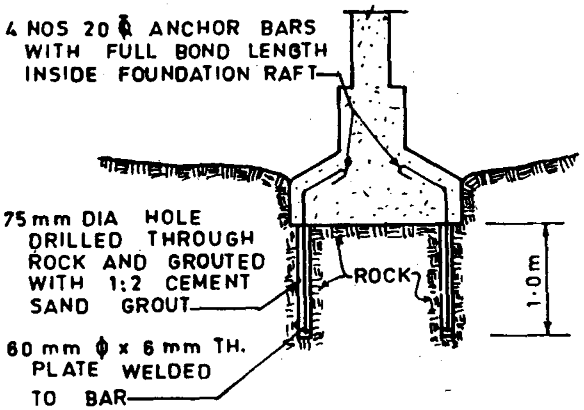


FIG.21·4 DETAILS OF ANCHORAGE OF FOUNDATION RAFT IN ROCK

Area of steel required to resist the uplift $= \frac{97{,}700}{200} = 490\ \text{mm}^2$

Use 4 Nos. 20 Φ on each longer side of the footing.

The details of anchorage of foundation raft are shown in Fig. 21.4.

21.4 PILE FOUNDATIONS

21.4.1 Where shallow spread or raft foundation is found unsuitable from the consideration of bearing power of the soil and where the possibility of scour of the shallow foundation is apprehended even though the foundation soil is otherwise suitable for taking the load, deep foundation is resorted to. If the depth of scour is not appreciable and if the underlying soil for pile foundation is suitable for taking the design load, pile foundations are adopted.

21.4.2 The pile foundations transmit the load into the underlying soils in such a manner that settlement of the foundations is not excessive and the shearing stresses in the soil are within the permissible limits after accounting for adequate factor of safety.

21.4.3 Piles may be classified into two groups depending on the manner by which they transmit the load into the soil viz. (1) friction piles and (2) end bearing piles. The former group of piles transmits the load into the soil through the friction developed between the entire pile surface of effective length and the surrounding soil whereas the latter group, if they are driven through very weak type of soil but resting on a very firm deposit such as gravel or rock at bottom, can transmit the load by end bearing only. Generally, in end bearing piles, some load is transferred to the soil by friction also. Similarly, in friction piles some load is transferred to the soil by end bearing also.

21.4.4 Type of Piles :

Piles are of various forms and of various materials. Most common types of piles used in the construction of highway bridges are :

(a) Timber piles

(b) Concrete piles

(i) Precast

(ii) Cast-in-situ

(c) Steel piles

(i) Tubular pile either empty or filled with concrete.

(ii) Screw piles.

2.4.5 Timber Piles

21.4.5.1 Timber piles are trunks of trees which are very tall and straight the branches being stripped off. Circular piles of 150 to 300 mm. diameter are generally used but squar piles sawn from the heartwood of bigger logs are sometimes utilised. For better performance during driving, the lengths of timber piles should not be more than 20 times diameter (or width). Common varieties of Indian timbers suitable for piles are Sal, Teak, Deodar, Babul, Khair etc.

21.4.5.2 Timber piles are cheaper than other varieties of piles but they lack in durability under certain conditions of service where variation of water level causing alternate drying and wetting of the piles is responsible for rapid decay of timber piles. If remain permanently under submerged soil, these piles may last for centuries without any decay. Timber piles may be used untreated or treated with chemicals such as creosote to prevent destruction by various bacteria or organism or decay. Timber piles are affected by marine borers in saline water.

21.4.6 Precast Concrete Piles

21.4.6.1 Precast concrete piles may be of square, hexagonal or octagonal shape, the former one being commonly used for their advantage of easy moulding and driving. Moreover, square piles provide more frictional surface which helps in taking more load. Hexagonal or octagonal piles, on the other hand, have the advantages that they possess equal strength in flexure in all directions and the lateral reinforcement may be provided in the form of a continuous spiral. Moreover, special chamfering of the corners is not required as in square piles.

21.4.6.2 Precast piles may be tapered or parallel sided with taper at the driving end only, the latter one is generally preferred. Sections of square piles vary with the length of the piles. Some common sections used are :

300 mm square for lengths up to 12 m.
350 mm square for lengths above 12 m up to 15 m.
400 mm square for lengths above 15 m up to 18 m.
450 mm square for lengths above 18 m up to 21 m.

Normally, the lengths of square piles are kept as 40 times the side for friction piles and 20 times the side for end bearing piles.

21.4.6.3 The precast piles are made of rich concrete mix of $1 : 1\frac{1}{2} : 3$ proportion, the pile head being made with richer mix of $1 : 1 : 2$ to resist the dynamic stresses during driving. Longitudinal reinforcement @ 1.5 percent to 3 percent of the cross-sectional area of the piles depending on length to width ratio and stirrups or lateral ties not less than 0.4 percent by volume are provided. Longitudinal bars should be properly tied by the lateral ties, the spacing of which should not be more than half the minimum width. The spacing of the lateral ties at the top and bottom of piles should be close and generally half the normal spacings. The reinforcement provided in precast piles are provided for resisting handling and driving stresses unless they are end bearing piles in which case the reinforcement provided in the piles transmit the load as in R. C. columns.

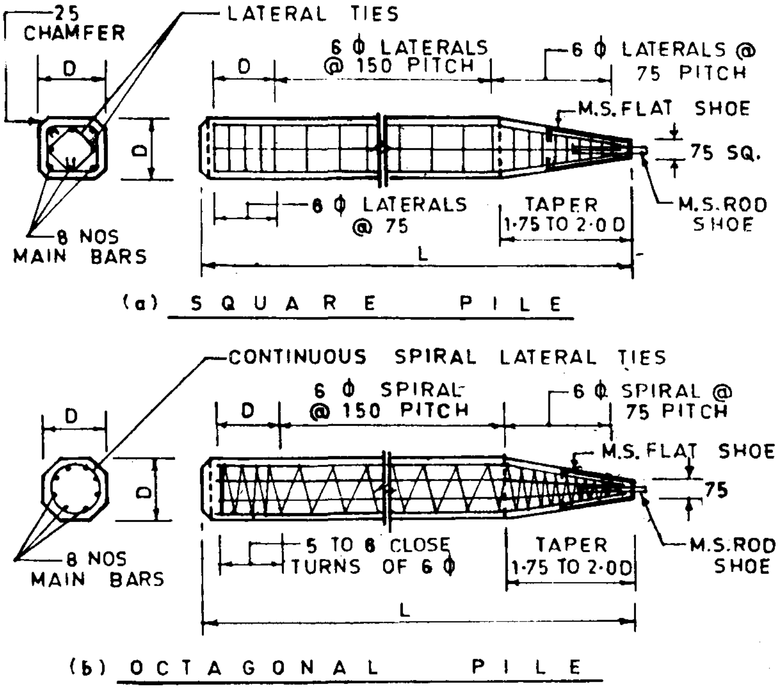


FIG. 21·5 PRECAST PILES

21.4.6.4 Handling and Lifting of Piles :

When précast piles are lifted, bending moment is induced in the piles due to the self weight of the piles for which reinforcement are required in the piles to cater for these handling stresses. To minimise the quantity of such reinforcement in piles, the lifting should be done in such a manner that the bending moments so developed should be brought to as minimum a value as possible. Two-point lifting of the piles is very common which may be outlined as follows.

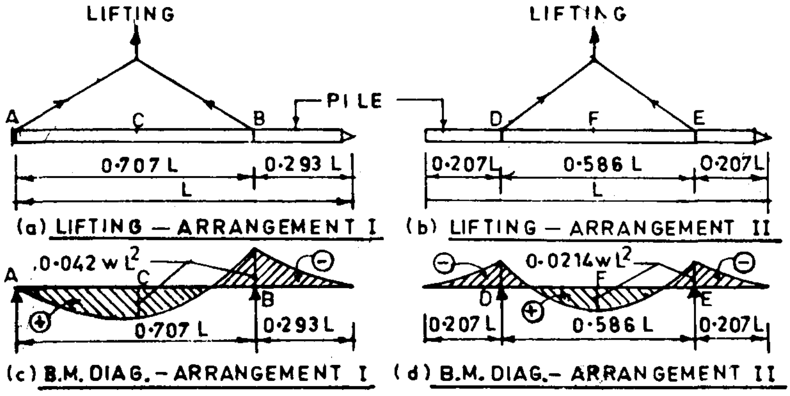


FIG. 21·6 DETAILS OF TWO-POINT LIFTING OF PILES

For the lifting arrangement as shown in Fig. 21.6(a) positive moment at C must be equal to the negative moment at B. Similarly, for the lifting arrangement as in Fig. 21.6(b) positive moment at F must be equal to the negative at D and E. To satisfy such moment condition, the dimensions of the lifting points must be as shown in the figure.

21.4.7 Cast-in-situ Concrete Piles (Driven or Bored)

21.4.7.1 There are many varieties of cast- in-situ piles but the main principle of making the piles is the same viz., a steel hollow pipe is either driven into or bored through the soil thus making a hollow cylindrical space into which the concrete is pored to form the cast-in-situ piles. Cast- in-situ piles are circular piles with variable size depending on the type and load carrying capacity. Simplex piles are normally of 350 to 450 mm diameter with load carrying capacity of 40 tonnes to 80 tonnes. Franki piles, on the other hand, are of 500 mm diameter and carry a load of 100 tonnes approx.

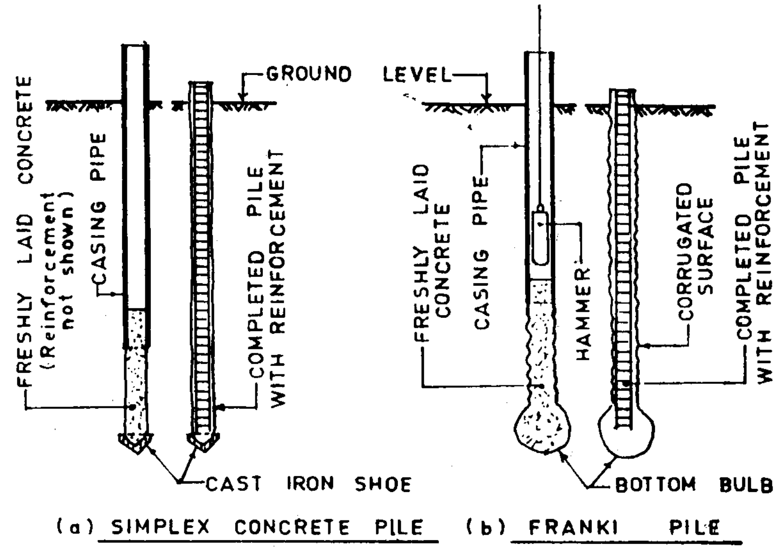


FIG. 21·7 CAST-IN-SITU DRIVEN PILES

21.4.7.2 In Simplex concrete piles, Fig. 21.7(a), a cast iron shoe is used at the bottom of the casing pipe to facilitate driving of the pipe by hammering at the top with an iron hammer over a wooden dolly. When final level is reached, reinforcement cage is lowered and the concrete is poured inside the pipe filling it partly. The pipe is slightly raised and again concrete is poured. This process is continued till concreting of the space is completed and the casing pipe is withdrawn leaving the completed cast-in-situ pile. This pile is mainly a friction pile but some load is taken by the tip of the pile also.

21.4.7.3 The driving procedure of the casing pipe in Franki piles [Fig. 21.7(b)] is slightly different from that in Simplex pile. Some dry concrete is poured into the pipe which is kept standing on the ground. This dry concrete forms a plug which is rammed by a hammer cylindrical in shape moving inside the pipe. The plug concrete grips the wall so tightly that the hammer forces down the pipe along with the plug concrete until the desired level is reached. At this level, the plug is broken, fresh concrete is poured and it is thoroughly rammed thus spreading the concrete to form a bulb which increases the bearing area of the pile at the bottom and helps in taking more load by bearing. As the tube is partly filled above the bulb after lowering the reinforcement cage, the tube is raised and the concrete is again rammed but with less violence than at the time of forming the bulb. This ramming makes the surface of the pile irregular in the form of corrugation which again increases the skin friction of the pile. The process is continued till the pile is completed. This sort of pile transmits the load by both friction and end bearing.

21.4.7.4 Vibro piles are quite similar to the Simplex type and the casing pipe is driven into the ground by hammering it at top and by providing a C.I. shoe at the bottom. The principal difference in this pile is that instead of filling the pipe with concrete in stages, it is completely filled with concrete of a fairly fluid consistency. During lifting of the casing pipe, a special type of hammer which hits an attachment of the pipe upwards is used. The vibration created by the hammer in the pipe and the static head of the fluid concrete helps to withdraw the pipe as well as to make a continuously vibrated shaft of the pile. The surface of this sort of piles is smooth and no corrugation is formed.

21.4.7.5 Bored Piles are found useful in places where the vibrations caused by the driving of the casing tube may be harmful to the neighbouring structures. These piles are cast in the hollow space made by removal of the earth by means of boring. Precautions should be taken to prevent the incoming of the earth into the casing. Bores should also be protected from necking caused by soft soil or piles should be protected during casting from loss of cement due to movement of subsoil water.

21.4.8 Tubular Steel Piles

21.4.8.1 Tubular piles may be driven open ended or with cast iron shoes as in casing pipe of cast-in-situ concrete piles. The piles when driven open ended are filled with soil automatically during driving. The piles with closed end may be kept empty or may be filled with concrete.

21.4.9 Screw Piles

21.4.9.1 A Screw pile consists of a circular steel shaft of various diameter ranging from 75 to 250 mm and ending in a large diameter screw blade at the bottom. The screw is a complete turn, the diameter of the blade being 150 mm to 450 mm. The base area of the screw piles is installed by screwing them down by means of Capstan with long bars fitted at the top of piles with the help of manpower. Electric motors are now-a-days employed for this purpose but the use of screw piles are becoming rarer day by day.

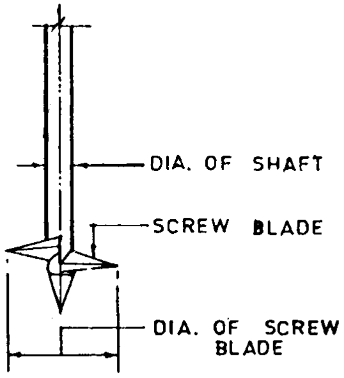


FIG.21·8 SCREW PILE

21.4.10 Pile Spacing

21.4.10.1 The recommended minimum spacing of friction piles is 3 *d*, where *d* is the diameter of circular piles or the length of the diagonal for square, hexagonal or octagonal piles. Further close spacing of the friction piles reduces the load bearing capacity of the individual pile and is, therefore, not economical. End bearing piles may be placed closer. No limit has been fixed for the maximum spacing of the piles but it does not generally exceed 4 *d*.

21.4.11 How Load Is Transferred Through Piles

21.4.11.1 ***Friction piles :*** When a load is placed on the top of a friction pile driven in granular or cohesive soil, it tends to penetrate further. This tendency of downward movement of the pile is resisted by the skin friction between the pile surface and the soil. The magnitude of the skin friction per unit area of pile surface depends on the value of normal earth pressure *p* and the coefficient of friction between the soil and the pile surface ; both of these values again depend on the nature of the pile surface and the nature of the soil.

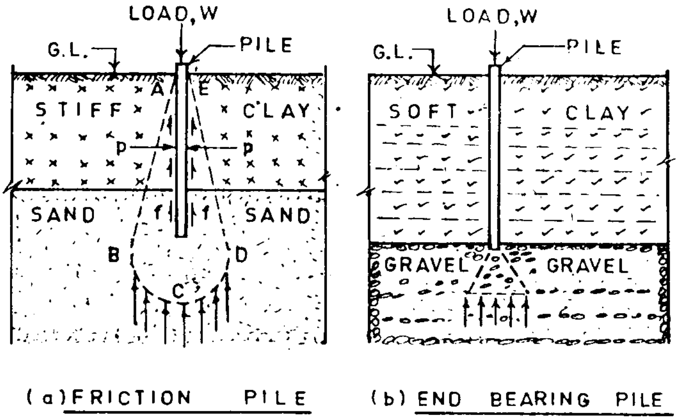


FIG. 21·9 LOAD TRANSFERENCE THRO' PILES (Dunham)[10]

21.4.11.2 *End Bearing Piles*

End bearing piles are driven through very poor type of soil to rest on firm base such as compacted sand or gravel deposits or rock. Therefore, the friction developed between the pile surface and the soil is practically very small and the whole load is transmitted by the pile through bearing. These piles act as columns and therefore, should be designed as such.

21.4.12 Evaluation of Ultimate Load Bearing Capacity of Piles from Soil Test Data-Static Formula

21.4.12.1 Piles in Granular Soils

The ultimate load carrying capacity, Q_u of piles in granular soil may be obtained from the following formula. A factor of safety of 2.5 shall be adopted for estimating the safe load carrying capacity of piles.

$$Q_u = Q_b + Q_f \qquad (21.4)$$

Where, Q_b = Ultimate base resistance and Q_f = Ultimate frictional resistance.

Again Q_b and Q_f are given by equations 21.5 and 21.6 respectively.

$$Q_b = A_b \times f_b = A_b\left(\frac{1}{2} d \cdot \gamma \cdot N_\gamma + P_D \cdot N_q\right) \qquad (21.5)$$

$$Q_f = \sum_{i=1}^{i=n} A_{si}\ f_{si} = \sum_{i=1}^{i=n} A_{si}\ K \cdot P_{Di}\ \tan\delta \qquad (21.6)$$

Where,

A_b	=	Plan area of the base of piles
f_b	=	Ultimate bearing capacity at the pile base
d	=	Diameter of pile in cm
γ	=	Effective unit weight of soil at pile toe in kg/cm^2

P_D = Effective over-burden pressure at pile toe in kg/cm^2. For piles longer than 15 to 20 times the pile diameter, maximum effective over-burden at the pile tip shall be limited to 15 to 20 times the pile diameter.

N_γ & N_q = Bearing capacity factor depending upon the angle of internal friction, ϕ at toe (Refer Table 21.1)

$\sum_{i=1}^{i=n}$ = Summation for n layers in which the pile is effective.

A_{si} = The surface area of pile in the 'i'th strata.

f_{si} = Average skin friction per unit area of the pile for the 'i'th strata.

K = Coefficient of earth pressure usually taken as 1 to 3 for driven piles and 1 to 2 for bored piles.

P_{Di} = Effective overburden pressure in kg/cm^2 in 'i'th layer (i varies from 1 to n below maximum scour level or effective level)

δ = Angle of wall friction between pile and soil in degrees (may be taken equal to ϕ)

TABLE 21.1 BEARING CAPACITY FACTORS, N_γ AND N_q *(BIS)*[4, 7]

Values of ϕ	Values of N_γ	Values of N_q
20°	5.39	10.0
25°	10.88	17.0
30°	22.44	28.0
35°	48.03	56.0
40°	109.41	130.0
45°	271.76	340.0

21.4.12.2 Piles in Cohesive Soils

The ultimate load carrying capacity, Q_u' of piles in purely cohesive soils may be determined from the following formula. A factor of safety of 2.5 shall be applied for getting the safe loads on piles.

$$Q_u^1 = A_b \cdot N_c \cdot C_b + \alpha \cdot C \cdot A_s \tag{21.7}$$

Where,

A_b = Plan area of the base of piles

N_c = Bearing capacity factor usually taken as 9.0

C_b = Average cohesion at pile tip in kg/cm^2

α = Reduction factor as given in Table 21.2

C = Average cohesion throughout the effective length of the pile in kg/cm^2

A_s = Surface area of pile shaft in cm^2

TABLE 21.2 : VALUES OF REDUCTION FACTOR, α (BIS)[5]

Consistency of soil	N Value	Value of α	
		For driven piles	For bored piles
Soft to very soft	< 4	1.0	0.7
Medium	4 to 8	0.7	0.5
Stiff	8 to 15	0.4	0.4
Stiff to hard	> 15	0.3	0.3

ILLUSTRATIVE EXAMPLE 21.2

Evaluate the safe bearing capacity of the bored piles 500 mm. dia and 22.0 m length embedded in a mixed type soil under a viaduct structure. The bore-log at the site of work is given below :

Depth below G.L.	Unit weight, γ	Values of	
		ϕ (degrees)	C (kg/cm^2)
0 — 5 m	1.8 t/m^3	10	0.25
5 — 10 m	-do-	10	0.25
10 — 15 m	-do-	10	0.25
15 — 20 m	-do-	15	0.20
20 — 25 m	-do-	20	0.15
25 — 30 m	-do-	30	0

Solution

Since the soil is mixed soil, the ultimate bearing capacity of the piles is estimated separately for the given values of ϕ and C.

Assuming the thickness of pile cap as 1.0 m and the top of pile cap 0.3 m below G. L., the depth of the pile tip is (0.3 + 1.0 + 22.0) = 23.3 m from G. L.

From equations, 21.4 to 21.6, $$Q_u = A_b\left(\frac{1}{2}d\cdot\gamma\cdot N_\gamma + P_D\cdot N_q\right) + \sum_{i=1}^{1=n} A_{si}K\cdot P_{Di}\cdot\tan\delta$$

Where,

A_b = $\frac{\pi}{4}\times(50)^2 = 1963.75$ cm^2

d = 50 cm

γ = Submerged weight of soil = (1.8 – 1.0) t/m^3 = 0.8×10^{-3} kg/cm^3

Assuming linear variation, the value of ϕ at pile tip at a depth of 23.3 m below G. L. = 26 degrees.

$\therefore$ N_γ and N_q from Table 21.1 are 13.19 and 19.2 respectively.

$P_D = \gamma \times$ depth of pile tip from G. L. $= 0.8 \times 10^{-3} \times 23.3 \times 100 = 1.86$ Kg/cm^2 but depth of overburden limited to 20d i.e., $P_D = 0.8 \times 10^{-3} \times 20 \times 50 = 0.8$ kg/cm^2

$\Sigma A_{si} = \pi \times 50 \times 22.0 \times 100 = 3{,}45{,}620$ cm^2; $\delta = \phi$ at top $= 10°$ and $\delta = \phi$ at tip of pile $= 26°$

Average value of $\tan \delta = \frac{1}{2}(0.1763 + 0.4877) = 0.33$; K = Average value of 1 and 2 i.e. 1.5

P_{Di} at mid depth $= 0.8 \times 10^{-3} \times \frac{23.3 \times 100}{2} = 0.93$ kg/cm^2

$$\therefore Q_u = 1963.75\left(\frac{1}{2} \times 50 \times 0.0008 \times 13.19\right) + 0.8 \times 19.2 + 3{,}45{,}620 \times 1.5 \times 0.93 \times 0.33$$

$$= 1963.75\,(0.2638 + 15.36) + 1{,}59{,}100 = 30{,}680 + 1{,}59{,}100 = 1{,}89{,}780 \text{ kg} = 189.78 \text{ tonnes.}$$

From equation 21.7, $Q_u^1 = A_b \cdot N_c \cdot C_b + \alpha \cdot C \cdot A_s$

A_b as before $= 1963.75$ cm^2; $N_c = 9.0$; $C_b = 0.15$ kg/cm^2

α from Table 21.2 may be taken as 0.5 ; C (average) for the entire depth $= 0.22$ kg/cm^2

A_s = As before $= 3{,}45{,}620$ cm^2

$\therefore$ $Q_u' = 1963.75 \times 9.0 \times 0.15 + 0.5 \times 0.22 \times 3{,}45{,}620 = 2651 + 38{,}018 = 40{,}669$ kg $= 40.67$ tonnes.

Total ultimate capacity $= Q_u + Q_u' = 189.78 + 40.67 = 230.45$ tonnes.

Therefore, applying a factor of safety of 2.5, the safe bearing capacity Q of the piles is $\frac{230.45}{2.5} = 92.18$ tonnes, say 90 tonnes.

21.4.13 Evaluation of Safe and Ultimate Load Bearing Capacity of Peirs from Driving Resistance — Dynamic Formula

21.4.13.1 This method takes into account the work done by the piles in overcoming the resistance of the ground during driving and as such equates the energy of the hammer blow. In some realistic methods, allowances for losses of energy due to the elastic compression of the piles and the soils are also made.

21.4.13.2 ***Formulae for Determining Safe Load R, on Piles (Engineering News Formulae)***

(a) For piles driven with freely falling drop hammer

$$R = \frac{16.7\,WH}{S + 2.54} \tag{21.8}$$

(b) For piles driven with single-acting steam hammer —

$$R = \frac{16.7\,WH}{S + 0.254} \tag{21.9}$$

(c) For piles driven with double acting steam hammer —

$$R = \frac{16.7\,H\,(W + A_p)}{S + 0.254} \tag{21.10}$$

Where,

R = Safe load on pile in kg.
W = Weight of hammer in kg.
H = Height of fall in metres.
S = Average penetration of the pile in cm per blow measured as the average of the last 5 to 10 blows under a drop hammer and 20 blows under a steam hammer.
A = Effective piston area in sq.cm.
p = Mean effective steam pressure in kg/cm^2

21.4.13.3 *Formula for Determining Ultimate Load R_u on Piles (Modified Hilley Formula)*

$$R_u = \frac{WHN}{S + \frac{C}{2}} \tag{21.11}$$

Where,

R_u = Ultimate resistance in tonnes. The safe load is obtained by applying a factor of safety of 2.5
W = Weight of hammer in tonnes
H = Height of fall in metres (Full value of H for trigger operated drop hammers, 0.9 H for single acting steam hammer and 0.8 H for winch operated drop hammers)
N = Efficiency of the blow i.e., the ratio of energy after impact to the striking energy of ram and is given by equations 21.12 & 21.13
S = Penetration per blow in cm.
C = Sum of the temporary elastic compression in cm of the pile, dolly, packings and ground as given in equation 21.14.

Where W is greater than P. E. and the pile is driven into penetrable ground,

$$N = \frac{W + P \cdot E^2}{W + P} \tag{21.12}$$

Where W is less than P. E. and the pile is driven into penetrable ground,

$$N = \frac{W + P \cdot E^2}{W + P} - \left[\frac{W - PE}{W + P}\right]^2 \tag{21.13}$$

Where,

P = Weight of pile, anvil, helmet etc. in tonnes
E = Co-efficient of restitution of the materials under impact and is equal to 0.5 for steel ram of double acting hammer striking on steel anvil and driving R.C. pile, 0.4 for cast iron ram of single acting or drop hammer striking on the head of R. C. pile, 0.25 for single acting or drop hammer striking a cap & helmet with hard wood dolly and driving R. C. piles.

The value of C in equation 21.11 is given by the following equation 21.14.

$$C = C_1 + C_2 + C_3 \tag{21.14}$$

Where,

C_1 = Temporary compression of dolly and packing.
C_2 = Temporary compression of piles, and
C_3 = Temporary compression of ground.

Again, the values of C_1, C_2 and C_3 may be obtained from the following formulae :

(i) Where the driving is without dolly and the cushion is about 2.5 cm thick

$$C_1 = 1.77 \frac{R_u}{A}$$

(ii) Where the driving is with short dolly up to 60 cm. long, helmet and cushion up to 7.5 cm thick —

$$C_1 = 9.05 \frac{R_u}{A}$$

(iii) $C_2 = 0.657 \dfrac{R_u \cdot L}{A}$;

(iv) $C_3 = 3.55 \dfrac{R_u}{A}$

Where,

R_u	=	Ultimate resistance of piles as per equation 21.11
L	=	Length of pile in metres.
A	=	Area of the pile in cm^2

21.4.14 Pile Grouping

21.4.14.1 ***Spacing of Piles*** — In case of piles founded on very hard stratum and deriving their load bearing capacity mainly from end bearing, minimum spacing of such piles shall be 2.5 times the diameter of piles. Friction piles derive their load bearing capacity mainly from friction and as such shall be spaced sufficiently apart since the cones of distribution or the pressure bulbs of adjacent piles overlap as shown in Fig. 21.11. Generally, the spacing of friction piles shall be minimum 3 times the diameter of piles.

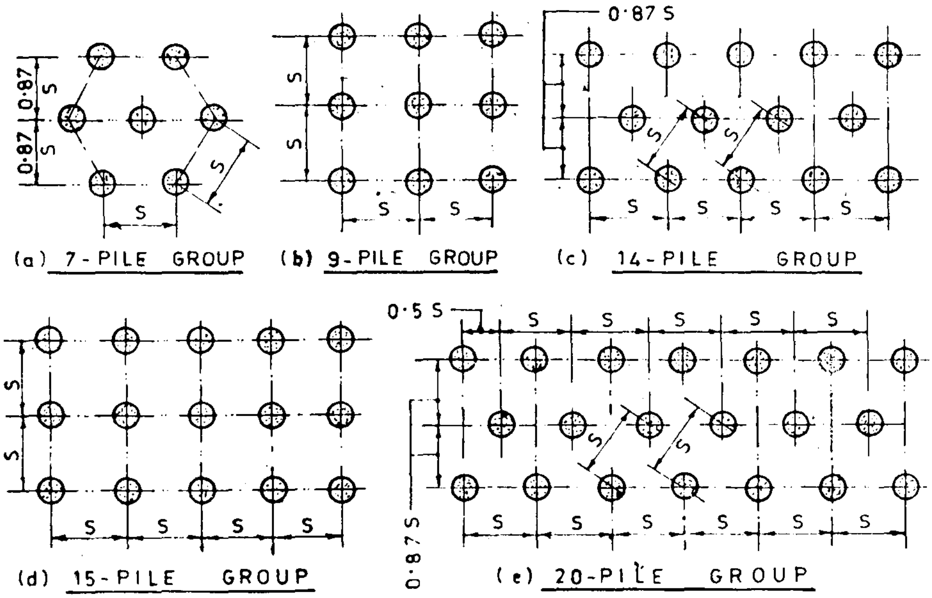


FIG. 21·10 TYPICAL ARRANGEMENT OF PILE GROUPING (Teng)[19]

21.4.14.2 ***Arrangement of Piles in a Group*** — Typical arrangement of piles in a group is shown in Fig. 21.10. The spacing S indicated in Fig. 21.10 shall be as recommended in Art. 21.4.14.1.

21.4.14.3 Group Action of Piles

(a) ***Pile Groups in Sands and Gravels :*** When piles are driven in loose sand and gravel, the soil around the piles up to a radius of at least three times the diameter of piles gets compacted. In such case, the efficiency of the pile group is more than unity. However, for practical purpose, the load carrying capacity of a pile group having N number of piles is N. Q_u, where Q_u is the capacity of individual pile. In case of bored piles in such soil strata, although no compaction effect exists, the group efficiency is also taken as unity.

(b) ***Pile Groups in Clayey Soils :*** In a group of friction piles in clayey or cohesive soil, the cones of distribution or the pressure bulbs of the adjacent piles overlap (Fig. 21.11-a) thus forming a new cone of distribution ABCDE (Fig. 21.11-b) the base area of which is much less than the sum of the areas of the cones of distribution of the individual pile before overlapping.

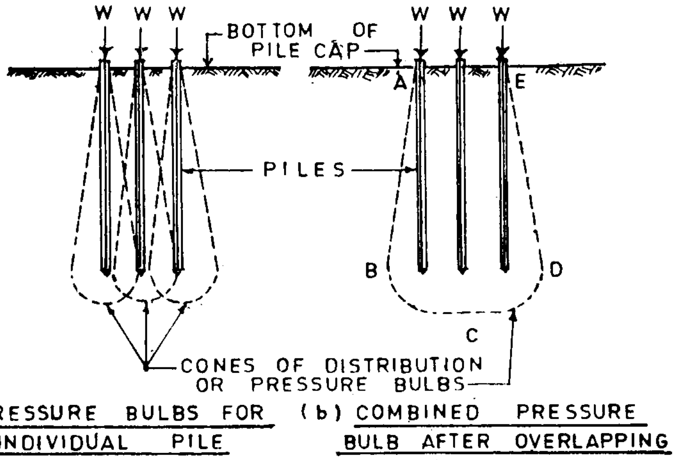


FIG.21·11 GROUP ACTION OF PILES (Dunham)[10]

The bearing area on which the loads from the piles are transferred through the cone of distribution is, therefore, less thus reducing the load carrying capacity of the individual pile due to group action. If the piles are driven with wider spacing, the overlapping of the cones of distribution will be lesser and therefore, the efficiency of the individual pile in that group will increase. It, therefore, transpires that increase in the nos. of piles in a pile group in which the cones of distribution overlap will not add anything to the load bearing capacity of the pile group since the soil has already attained the ***"saturated"*** condition. Friction piles in clayey soils may, therefore, fail either individually or as a block. The ultimate load bearing capacity Q'_{gu}, of the block (Fig. 21.12) is given by :

$$Q'_{gu} = C_b \cdot N_c \cdot A_g + P_g \cdot D \cdot C \qquad (21.15)$$

Where,

Q'_{gu} = Ultimate bearing capacity of the block in kg.
C_b = Average cohension at pile tip in kg/cm^2
N_c = Bearing capacity factor usually taken as 9.0
A_g = Area of the block in sq.cm. measured by outer dimensions of the pile group

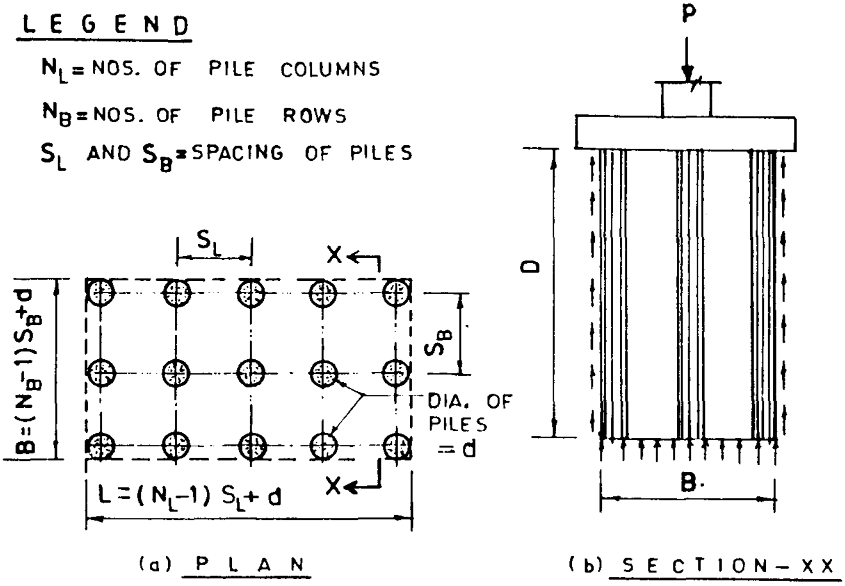


FIG. 21·12 BLOCK FAILURE OF PILE GROUP IN COHESIVE SOILS (Teng)[19]

P_g	=	Perimeter of the block in cm. measured by the outer dimensions of the pile group $= 2(L + B) \times D$.
D	=	Length of the block i.e., the length of the piles in cm.
C	=	Average cohesion throughout the block in kg/cm^2

Since the block is to sustain its self weight in addition to the loads from the piles, the safe load of the block shall be calculated after deducting the self weight of the block. Normally, a factor of safety of 3 is allowed over Q'_{gu} to get the safe load carrying of the block. Therefore, the safe load carrying capacity of the pile group

$$= \frac{Q'_{gu}}{3} - \gamma \cdot A_g \cdot D \tag{21.16}$$

ILLUSTRATIVE EXAMPLE 21.3

A pier foundation for a medium span bridge is supported on a group of cast-in-situ bored piles as shown in Fig. 21.13 driven through clayey soil. The relevant data are given below :

(i) ***Length of pile below maximum scour (which is very small in this case) = 25 m.***

(ii) ***Diameter of piles, d = 500 mm.***

(iii) ***Average cohesion throughout the length of the piles, C = 0.45 kg/cm²***

(iv) ***Average cohesion at pile tip, C_b = 0.5 kg/cm²***

(v) ***Angle of internal friction, $\phi = 0$***

Determine whether individual capacity of piles or the block capacity governs the design if the pile spacing is (a) 3d and (b) 2.5 d.

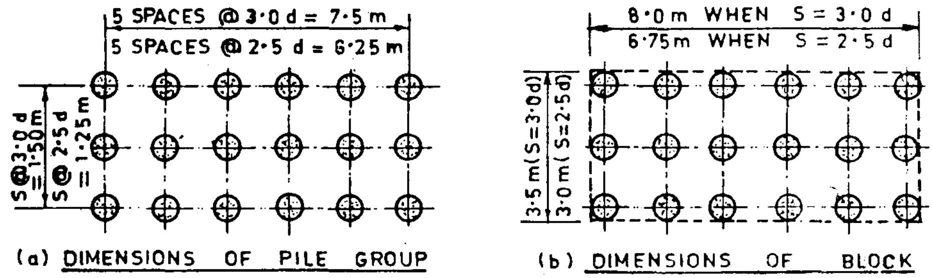


FIG. 21·13 (Example 21·3)

Solution :

(a) ***Criteria for failure***

(i) *Capacity of individual piles*

From equation 21.7, $Q_u^1 = A_b \cdot N_c \cdot C_b + \alpha \cdot C \cdot A_s = \frac{\pi}{4}(50)^2 \times 9.0 \times 0.5 + 0.5 \times 0.45 \times \pi \times 50 \times 25 \times 100$

$= 8{,}835 + 88{,}370 = 97{,}205 \text{ kg} = 97.2 \text{ tonnes}$

With a factor of safety of 2.5, safe load on each pile $= \frac{97.2}{2.5} = 38.88$ tonnes.

$\therefore$ Total load carrying capacity of the group (group efficiency not considered) $= 18 \times 38.88 = 699.84$ tonnes, say 700 tonnes.

(ii) *Capacity of the block when S = 3d*

From equation 21.15, $Q'_{gu} = C_b \cdot N_c \cdot A_g + P_g \cdot D \cdot C$

From Fig. 21.13(b), $A_g = 8.0 \text{ m} \times 3.5 \text{ m} = 28.0 \text{ m}^2 = 28 \times 10^4 \text{ cm}^2$
$P_g = 2(8.0 + 3.5) = 23.0 \text{ m} = 2300 \text{ cm}.$

$\therefore$ $Q'_{gu} = 0.5 \times 9.0 \times 28 \times 10^4 + 2300 \times 25 \times 10^2 \times 0.45$
$= (126 \times 10^4 + 258.75 \times 10^4)$ Kg. $= 3847.5$ tonnes.

$\therefore$ Safe load carrying capacity of the block Q_{gu} with a FOS of $3 = \frac{Q'_{gu}}{3} = \frac{3847.5}{3} = 1282.5$ tonnes

Self weight of the block considering submerged weight of soil, $\gamma = 800 \text{ kg/m}^3$

$$\gamma \cdot A_g \cdot D = 800 \times 10^{-6} \times 28 \times 10^4 \times 25 \times 10^2$$

$= 560{,}000 \text{ kg} = 560$ tonnes.

$\therefore$ Safe load of the pile group $= 1282.5 - 560 = 722.5$ tonnes. This is greater than the total capacity of all the piles viz. 700 tonnes. Hence the individual capacity of the piles will govern the design.

(iii) *Capacity of the block when S = 2.5 d*

From Fig. 12.13 (b), $A_g = 6.75 \times 3.0 = 20.25 \text{ m}^2 = 20.25 \times 10^4 \text{ cm}^2$

$P_g = 2(6.75 + 3.00) = 19.5 \text{ m} = 1950 \text{ cm}.$

$\therefore$ $Q^1_{gu} = 0.5 \times 9.0 \times 20.25 \times 10^4 + 1950 \times 25 \times 10^2 \times 0.45 = (91.12 + 219.37) \times 10^4 \text{ kg} = 3104.9$ tonnes

Safe load carrying capacity of the block, $Q_{gu} = \frac{Q'_{gu}}{3} = \frac{3104.9}{3} = 1035$ tonnes

Self weight of the block $= \gamma \cdot A_g \cdot D = 800 \times 10^{-6} \times 20.25 \times 10^4 \times 25 \times 10^2 = 405{,}000$ kg $= 405$ tonnes.

$\therefore$ Safe load of the pile group = 1035 – 405 = 630 tonnes.

This is less than the total capacity of all the piles viz., 700 tonnes. Hence in this case, the capacity of the block governs the design. The group efficiency in this case is $\frac{630}{700} \times 100 = 90$ percent. Therefore, by reducing the pile spacing in clayey soils from 3d to 2.5d in this particular case, the efficiency of the individual pile in the pile group is 90 percent.

21.4.15 Lateral Resistance of Piles

21.4.15.1 Piles driven under the abutments or retaining walls are always subjected to horizontal forces in addition to the vertical loads on them. These horizontal forces are resisted by the lateral resistance of the piles. Failure of the structure on account of the horizontal forces may be due to —

(i) Shear failure of the pile itself

(ii) Failure of the pile by bending

(iii) Failure of the soil in front of the piles thus causing tilting of the structure as a whole.

21.4.15.2 The section of and reinforcement for the piles should be such as to resist both the shear and the bending coming on the piles. Tendency of tilting of the structure as a whole is resisted by the passive resistance offered by the soil in front of the piles.

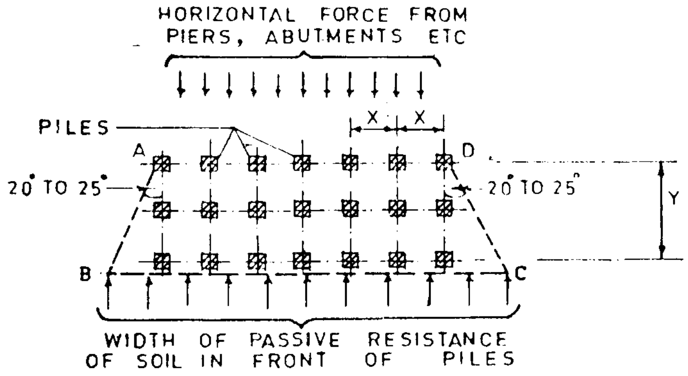


FIG.21·14 PASSIVE RESISTANCE OF SOIL IN FRONT OF PILES (Dunham)[10]

21.4.15.3 It has been observed that the distance between the outermost piles in front row of the pile group plus some additional distance due to dispersion effect (which may be taken as 20° to 25° as shown in Fig. 21.14) is effective in offering the passive resistance to the movement of the piles along with the structure supported on them. Thus from Fig. 21.14, the width BC in front of pile group offering passive resistance may be given by the formula.

$$BC = AD + 2y \tan 20°$$

$$= (n-1)x + 2y \tan 20° \qquad (21.17)$$

Where, n = nos. of piles in front row.

Generally, 3.0 m. to 4.5 m. top length of the piles below the level reliably protected or the maximum scour depth is effective in offering the passive resistance. Thus, whenever pile groups are subjected to horizontal forces, the area in front as given by the width BC and a depth of about 3.0 m. to 4.5 m. offers the passive resistance against movement of the structure. In addition, the horizontal resistance of the pile cap, if remains in contact with soil, may also be considered.

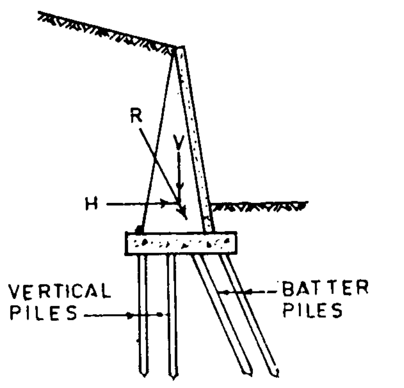


FIG. 21·15 BATTER PILES

21.4.16 Batter Piles

21.4.16.1 In high abutments, retaining walls etc. where the magnitude of the horizontal force acting on the piles is such that the lateral resistance of vertical piles is insufficient to resist it, batter piles or raker piles are the correct answer to such problems. The disadvantage is that to drive such piles, special skill and special type of driving equipments are required.

21.4.16.2 The horizontal component of the batter pile takes the horizontal load along with the horizontal resistance of the base of pile cap if remains in contact with the soil and therefore, the use of batter piles increases the factor of safety against sliding and overturning. Regarding vertical load bearing capacity of batter piles, it is generally assumed that the batter piles carry the same amount of vertical loads as vertical piles do.

21.4.17 Evaluation of Loads on Piles

21.4.17.1 If the foundation is subjected to direct load only, the load on the pile is obtained by dividing the load with the number of piles.

When the foundation is subjected to a moment in addition to the direct load, the load on piles may be determined as per equation 21.18 below which is quite analogous to equations 21.1 and 21.2.

$$\text{Load on piles} = \frac{W}{n} \pm \frac{M \cdot y}{I} \qquad (21.18)$$

Where,

W = total load
n = nos. of piles
y = distance of the pile under consideration from the c.g. of the pile group.
I = Moment of inertia of the pile group about an axis through the c.g. of the pile group.

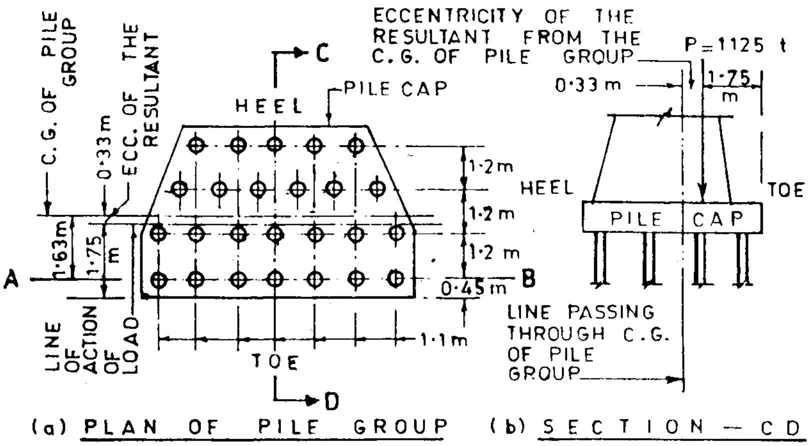
21.4.17.2 In calculating the moment of inertia of the pile group, piles are assumed as units that are concentrated in their longitudinal centre lines, the moment of inertia of the piles about their own centre being neglected.

ILLUSTRATIVE EXAMPLE 21.4

A group of precast piles is subjected to an eccentric resultant load of 1125 tonnes as shown in Fig. 21.16(b). Calculate the maximum and minimum load carried by the piles.

Solution

$$\text{Distance of c.g. of pile group from AB} = \frac{7\times 0 + 7\times 1.2 + 6\times 2.4 + 5\times 3.6}{7+7+6+5} = \frac{40.8}{25} = 1.63 \text{ m.}$$



(a) PLAN OF PILE GROUP (b) SECTION — CD

FIG. 21·16 LOAD ON PILES (Example 21·4)

∴ Eccentricity of the resultant from the c.g. of the pile group = 1.63 – (1.75 – 0.45) = 0.33 m.

∴ Moment of the resultant load about c.g. of pile group = 1125 × 0.33 = 371.25 tm.

Moment of inertia of the pile group about its c.g.

$$I = 7 \times (1.63)^2 + 7 \times (0.33)^2 + 6 \times (0.87)^2 + 5 \times (2.07)^2 = 18.60 + 0.76 + 4.54 + 21.42 = 45.32 \text{ m}^2$$

From equation 21.18, maximum load on piles (on toe side) $= \dfrac{W}{n} + \dfrac{My}{I} = \dfrac{1125}{25} + \dfrac{371.25 \times 1.63}{45.32}$

$= 45.00 + 13.35 = 58.35$ tonnes per pile.

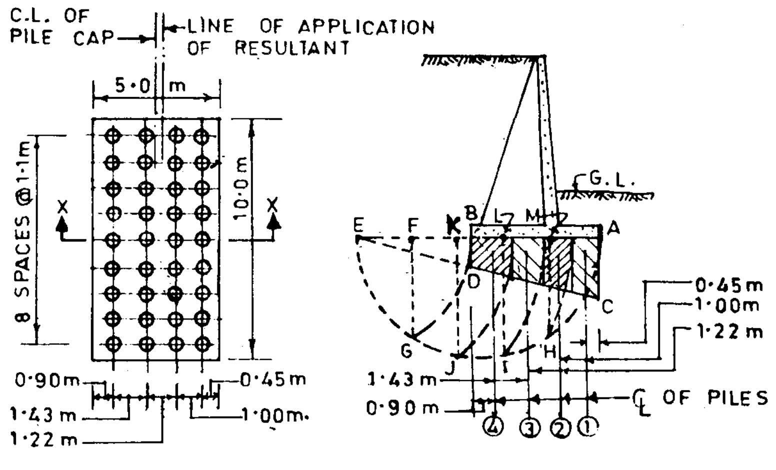
Minimum load on piles (on heel side) $= \dfrac{W}{n} - \dfrac{My}{I} = \dfrac{1125}{25} - \dfrac{371.25 \times 2.07}{45.32}$

$= 45.00 - 16.96 = 28.04$ tonnes per pile

21.4.18 Equal Load on Piles with Eccentric Loading

21.4.18.1 The loads carried by the piles on the toe and the heel side are found to be different (Illustrative Example 21.4) because though due to eccentric load from the superstructure, the ground reaction per unit area on the toe side is more than that on the heel side, the area of the foundation covered by each pile is the same and therefore, total ground reaction of the area covered by each pile, i.e., load carried by each pile on the toe side is more than that on the heel side. From practical as well as soil strata consideration, it is difficult to make different length of piles for the toe and heel side. But adoption of same spacing of piles for heel side as that for toe side is uneconomical when length of piles remains the same.

21.4.18.2 From the consideration of economy, it is desirable to adjust the pile spacings in such way that load shared by each pile in pile foundations subjected to direct load and moment i.e., subjected to eccentric load is equal. A graphical method for this is described below by Illustrative Example 21.5.



(a) PLAN (b) SECTION — XX

FIG. 21·17 GRAPHICAL METHOD OF PILE SPACING FOR EQUAL LOAD ON PILES (Ex. 21·5)

ILLUSTRATIVE EXAMPLE 21.5

In a retaining wall 10 m long, a resultant vertical load of 800 tonnes acts with an eccentricity of 0.33 m. from the centre line of the pile cap towards toe side. Determine the pile spacing so as to get equal load on each pile. The piles may be assumed to bear a load of 25 tonnes per pile.

Solution

Load per running metre of wall $= \dfrac{800}{10.0} = 80$ tonnes.   Eccentricity = 0.33 m.

$\therefore$ Moment about the centre line of the pile cap per metre = $80 \times 0.33 = 26.4$ tm.

Section modulus of the pile cap per metre length of wall $= \dfrac{1 \times (5.0)^2}{6} = 4.17\ m^3$

$\therefore$ Maximum and minimum foundation pressure $= \dfrac{P}{A} \pm \dfrac{M}{Z} = \dfrac{80}{5.0 \times 1.0} \pm \dfrac{26.4}{4.17} = 16.0 \pm 6.33$

$= 22.33\ t/m^2$ or $9.67\ t/m^2$

The foundation pressure diagram ACDB is drawn to scale with the above values of maximum and minimum foundation pressures [Fig. 21.17(b)]. AB and CD are produced to meet at E. With AE as diameter, semicircle AHIJGE is drawn. The arc BG is drawn with E as centre. From G, FG is drawn perpendicular on AE. AF is divided into "n" equal lengths where n is the nos of rows of piles required within the width AB.

In the example, total load per metre = 80 tonnes. Assuming 1.1 m pile spacing in the longitudinal direction,

load per 1.1 m length of wall = $80 \times 1.1 = 88$ tonnes

$\therefore$ Nos. of piles required per row $= \dfrac{88}{25} = 3.52$, Say 4.

Therefore, AF is divided into four equal lengths viz., AM, ML, LK and KF. From these points on AF, perpendiculars are dropped to meet the semicircle at H, I and J. With E as centre and EH, EI, EJ as radius, arcs are drawn to meet the line AB dividing the pressure diagram into four parts the area of which is the same and therefore, the pile provided to cater for the foundation pressure of each such area will carry equal load. The pile centre line will be the line through the centroid of the above trapezoidal pressure diagrams. The spacings of the piles to have equal load are scaled off and are shown in the Fig. 21.17(a). The actual load shared by each pile with the above spacing is calculated below to show the accuracy of the method.

$$\text{Distance of centroid of pile group from A} = \frac{(1 \times 0.45 + 1 \times 1.45 + 1 \times 2.67 + 1 \times 4.10)}{4} = 2.17 \text{ m.}$$

Point of application of the resultant load from A = 2.5 – 0.33 = 2.17 m.

Hence, the eccentricity of the resultant with respect to the centroid of pile group is nil and the load shared by each pile is equal, the load per pile being $\frac{800}{36}$ = 22.22 tonnes per pile.

21.4.19 Driving of Piles

21.4.19.1 Piles are driven by means of either drop hammer or steam hammer. The hammer is supported by a special frame known as pile-driver which consists of a pair of guides. The hammer moves within the guides and falls from the top of the guide on the top of the piles to be driven. The hammer which is lifted by manual labour or by mechanical power and is then released to fall freely by gravity is known as drop-hammer. Now-a-days steam hammers are used for pile driving. The steam hammer which is lifted by the steam-pressure and is then allowed to fall freely is a single acting steam hammer but the one which is also acted on by the steam-pressure during downward movement and adds to the driving energy is known as double acting steam hammer.

21.4.20 Load Test on Piles

21.4.20.1 The pile formulae both, static and dynamic, given in the previous articles predict approximately the safe load the piles will carry but it is always desirable to verify the load carrying capacity of the piles by load tests.

21.4.20.2 ***Initial Tests and Routine Tests :*** There shall be two categories of test piles, viz., initial tests and routine tests. Initial tests are carried out on test piles at the beginning prior to driving of working piles to determine the length of piles to sustain the design load. Initial test shall be carried out on minimum two piles. Routine tests are carried out on working piles to verify the capacity of piles as obtained by initial tests. While initial tests may be conducted on single pile, the routine tests may be carried out on single pile or a group of piles, two to three in number. The latter is preferable since the load carrying capacity of piles in a group is less specially in clayey soils and mixed soils. Routine tests shall be carried out on 2 percent of the piles used in the foundation.

21.4.20.3 ***Procedure for Vertical Load Tests :*** The test load may be applied in stages directly over a loading platform as shown in Fig. 21.18 or by means of hydraulic jack with pressure gauge and remote control pump, reacting against a loading platform similar to Fig. 21.18. The difference between the former and the latter method is that while all the test load placed on the platform is transferred on the test piles in the former method, the reaction of the jack is only transferred as load on the piles in the latter

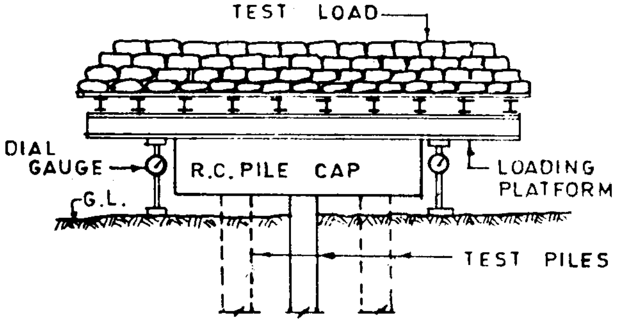


FIG. 21·18 LOAD TEST ON PILES

method though the load on the platform normally exceeds the required reaction. Pile testing by reaction method may also be done by taking advantage of the adjacent piles which give the required jack reaction by negative friction. For testing of piles by direct loading method, R. C. pile caps are usually provided on the top of piles for using it as loading platform as well as for transferring the load on the piles uniformly.

21.4.20.4 *Procedure for Lateral Load Tests on Piles*

Lateral load tests may be conducted by jack reaction method with the hydraulic jack and gauge in between two piles or two groups of piles. The reaction of the jack as indicated by the gauge is the lateral resistance of the pile or the pile group.

21.4.20.5 *Application of Test Loads, Measurement of Displacements and Assessment of Safe Loads for Vertical Load Tests*

(a) *For Initial Load Test :* The test loads shall be applied in increments of about 10 percent of the test loads and measurements of displacements shall be done by three dial gauges for single pile and four dial gauges for a group of piles. Each stage of loading shall be maintained till the rate of settlement is not more than 0.1 mm per hour in sandy soils and 0.02 mm per hour in clayey soils or a maximum of 2 hours whichever is greater. The loading shall be continued up to the test load which is twice the safe load (safe load as estimated by using static formula — Art. 21.4.12) or the load at which the total displacement of the pile top equals the following specified value :

The safe load on single pile shall be the least of the following :

(i) Two-third of the final load at which the total settlement attains a value of 12 mm.

(ii) Fifty percent of the final load at which the total settlement equals 10 percent of the pile diameter.

The safe load on groups shall be the least of the following :

(i) Final load at which the total settlement attains a value of 25 mm.

(ii) Two-third of the final load at which the total settlement attains a value of 40 mm.

(b) *For Routine Load Tests :* Loading shall be carried out to one and half times the safe load or up to the load at which the total settlement attains a value of 12 mm for single pile and 40 mm for group of piles whichever is earlier. The safe load shall be given by the following :

(i) Two-third of the final load at which the total settlement attains a value of 12 mm for single pile.

(ii) Two-third of the final load at which the total settlement attains a value of 40 mm for a group of piles.

21.4.20.6 *Loading etc. for Lateral Load Tests*

The loading shall be applied in increments of about 20 percent of the estimated safe load after the rate of displacement is 0.5 mm per hour in sandy soils and 0.02 mm in clayey soils or 2 hours whichever is earlier. The safe lateral loads shall be taken as the least of the following :

(a) 50 per cent of the total load at which the total displacement is 12 mm at the cut off level.

(b) Total load at which the total displacement is 5 mm at the cut-off level.

21.4.20.7 *Pull-out Tests on Piles*

For this test, clause 4.4 of *"IS:2911 (Part IV) — 1979 : Code of Practice for Design and Construction of Pile Foundations — Load Tests on Piles"* shall be referred.

21.4.20.8 *Cyclic Load Tests & Constant Rate of Penetration Tests*

Details of these tests are given in Appendix A and Appendix B of *"IS:2911 (Part IV) — 1979"* respectively which may be referred in this regard.

21.4.21 Pile-Cap

21.4.21.1 R. C. Pile – caps of adequate thickness are required to be provided on the top of piles to transfer the load from the structure on to the piles. The pile- caps are designed on the following principles :

(i) Punching shear due to load on the piers or columns or on the individual piles.

(ii) Shear at pier or column face.

(iii) Bending of the pile cap about the pier or column face.

(iv) Settlement of one row of piles and the consequent bending and shear of the pile cap.

21.4.21.2 An off-set of 150 mm shall be provided beyond the outer faces of the outermost piles in the group. When the pile cap rests on ground, a mat concrete (1:4:8) of 80 mm thickness shall be provided at the base of the pile cap. The top of pile shall be stripped of concrete and the reinforcement of the pile shall be adequately anchored into the pile cap for effective transmission of the loads and moments to the ground through the piles. At least 50 mm length of the pile top after stripping of concrete shall be embedded into the pile cap. The clear cover for main reinforcement shall not be less than 60 mm.

21.4.22 Pile Reinforcement

21.4.22.1 The area of longitudinal reinforcement in precast piles shall be as below to withstand the stresses due to lifting, stacking and transport.

(i) 1.25 percent for piles having a length less than 30 times the least width.

(ii) 1.5 percent for piles having a length greater than 30 and up to 40 times the least width.

(iii) 2.0 percent for piles having a length exceeding 40 times the least width.

21.4.22.2 The area of longitudinal reinforcement in driven cast-in-situ and bored cast-in-situ concrete piles shall not be less than 0.4 percent of the shaft area.

21.4.22.3 Lateral reinforcement in piles shall not be less than 0.2 percent of the gross volume in the body of the piles and 0.6 percent of the gross volume in each end of the pile for a distance of about 3 times the least width or diameter of the piles. The minimum dia. of the lateral reinforcement shall be 6 mm.

21.5 WELL FOUNDATIONS

21.5.1 Where pile foundations are unsuitable due to site conditions, the nature of the soil strata or for the reason of comparatively deep scour, well foundations are adopted. The components of a well are shown in Fig. 21.19 and described in Art. 21.5.2 to 21.5.6.

21.5.2 Cutting Edge and Well Curb

21.5.2.1 At bottom, wells are provided with a steel cutting edge made of m.s. plates and angles rivetted or welded together and anchored into the well curb by means of anchor bars. Concrete well curbs are triangular in section in order to assist in removing the earth by grabbing and to help easy sinking of the wells. The inclination of the well curb should not exceed 35 degrees with the vertical. These curbs are properly reinforced so as to make it strong enough to resist the stresses during sinking. Usually reinforcement both in the form of stirrups and longitudinal bars are provided not less than 72 kg. per cu.m. excluding bond rods of steining. Link bars are used to keep the longitudinal bars and stirrups in position. The concrete to be used in the well curbs shall generally be of grade M20.

21.5.2.2 Where pneumatic sinking is to be adopted, the internal angle of the well curbs shall be steep enough for easy access of the pneumatic tools. In case, blasting is to be resorted to sink the wells, the full height of the

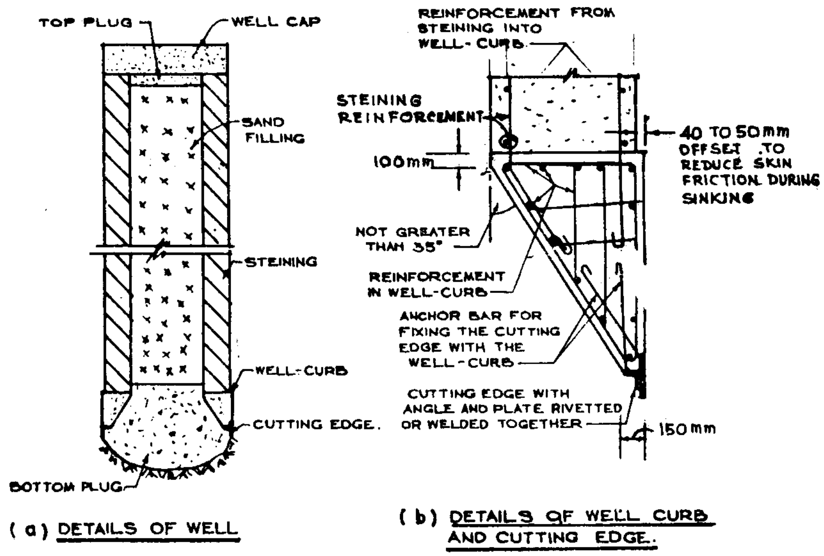


FIG.21·19 COMPONENTS OF WELL FOUNDATION

internal face and half height of the external face of the curb shall be protected with m.s. plate of 6 mm thickness properly anchored to the curb by anchor bars.

21.5.3 Steining

21.5.3.1 The steining is made of brick or stone masonry or of mass concrete. Nominal reinforcement shall not be less than 0.12 percent of gross sectional area of steining to resist the tensile stress that may be developed in the well steining in case top portion of the steining is stuck to a layer of stiff clay and the remaining portion is hung from top. Two layers of vertical steining bars with binders are preferred to one central layer only.

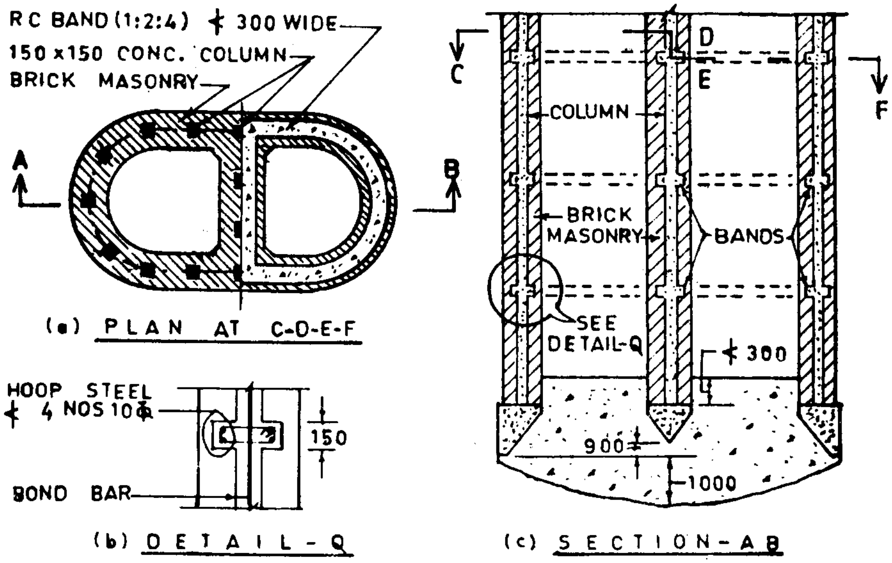


FIG. 21·20 DETAILS OF BRICK MASONRY WELLS (IRC)[3]

21.5.3.2 In case of brick steining, vertical bond rods shall be provided at the middle of the steining at a rate not less than 0.1 percent of the gross steining area. These bars shall be encased with concrete of M20 grade within a column of 150 × 150 size. These columns shall be tied with R. C. bands of suitable width not less 300 mm and of 150 mm depth. The spacing of such bands shall be 3 m or 4 times the thickness of the steining whichever is less (Fig. 21.20).

21.5.4 Bottom Plug

21.5.4.1 When the sinking is completed and the founding level is reached, the wells after making the necessary sump are plugged with 1:2:4 concrete. This is usually to be done under water for which special type of equipments are to be used in order to protect the concrete from being washed away when taken through water. For this purpose, two methods are commonly used. The first method is known as ***"Chute method"*** or ***"Contractor's method"*** in which some steel pipes usually known as tremie 250 mm to 300 mm diameter with funnel at top are placed inside the wells. The top of these pipes is kept above water level and the bottom at the bottom level of well. The concrete, when poured in the funnel, moves downwards due to gravity and reaches the bottom. The pipes are shifted sideways as the concreting proceeds.

21.5.4.2 In the second method, a more or less water-tight box is used for under-water concreting. The bottom of the box is made such that when the box reaches the plugging level, the bottom of the box is opened downwards by releasing a string from above and the concrete is placed at the bottom of the well. This method is known as ***"Skip box"*** method.

21.5.4.3 The function of the bottom plug is to distribute the load from the piers and abutments on to the soil strata below through the well steining. The load from the piers and abutments distributed over the well-cap and then to the well steining finally reaches the well curb. Having a tapered side in contact with bottom plug, the load from the curb is ultimately transferred to the bottom plug and then onto the soil below. For better performance, the bottom plug shall have adequate thickness as shown in Fig. 21.20(c).

21.5.5 Sand Filling

21.5.5.1 The well pockets are usually filled with sand or sandy clay but sometimes the pockets are kept empty to reduce the dead load of well on the foundation. It is desirable that at least the portion of the pockets below maximum scour level should be filled with sand for stability of the wells. In each case, a top plug is provided over the sand filling.

21.5.6 Well-Cap

21.5.6.1 Load from the piers and abutments are transferred to the well-steining through the well-caps which should, therefore, be reinforced adequately to withstand the resulting stresses caused by the superimposed loads and moments.

21.5.7 Shapes of Wells

21.5.7.1 Wells of various shapes are used depending on the type of soil through which they are to be sunk, the type of pier to be supported and the magnitude of the loads and moments for which they are to be designed. The following shapes, as shown in Fig. 21.21 and described in Art. 21.5.7.2 to 21.5.7.4, are very common :

21.5.7.2 Double-D, octagonal or dumb-bell shaped wells have generally twin poekets or dredge holes due to which greater control over the shifts and tilts of wells is possible. In addition, dumb-bell shaped wells offer greater resistance to tilting in the longitudinal direction but while brick or concrete can be used in the construction of well- steining in both the double-D or octagonal wells, laboar cost is more if brick-steining is used in dumb-bell wells.

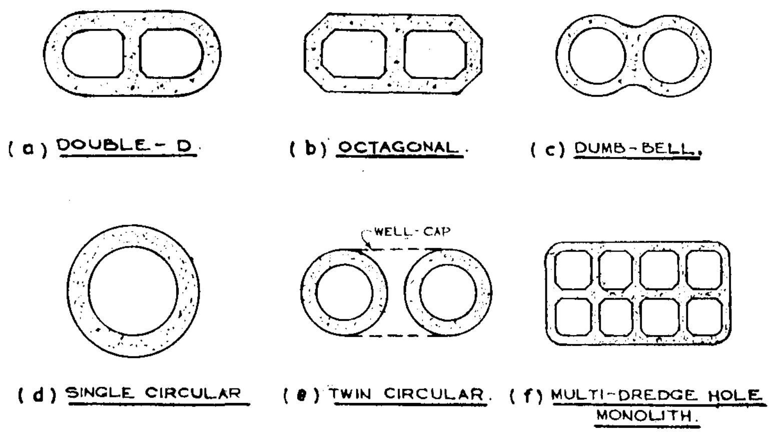


FIG.21·21 VARIOUS SHAPES OF WELLS

21.5.7.3 Single circular wells are most economical where the moments in both the longitudinal and transverse directions are more or less equal. Moreover, for the same base area, these wells have lesser frictional surface on account of which lesser total sinking effort is required to sink the wells. Twin-circular wells are more or less similar to single circular wells but these are suitable where the length of pier is more but twin-circular wells are not favoured where possibility of differential settlement between the two wells is not over-ruled. Both brick and concrete may be used in the steining of circular wells.

21.5.7.4 Multi-dredge hole wells or monoliths are adopted in supporting piers or towers of long span bridges. This sort of monoliths was used in supporting the main towers of Howrah Bridge at Calcutta (Fig. 17.8). The size of the monolith was 55.35 m × 24.85 m with 21 dredging shafts each 6.25 m square.

21.5.8 Depth of Wells

21.5.8.1 In deciding the founding levels of wells, the following points should be duly considered :

(i) The minimum depth of well is determined from the considerations of maximum scour so as to get the minimum grip length below the maximum scour level for the stability of the well as described in Art. 3.13 in Chapter 3.

(ii) The foundation may have to be taken deeper if the soil at the founding level as determined from the preceding para is not suitable to bear the design load.

(iii) Passive resistance of earth on the outside of well is taken advantage of in resisting as far as possible the external moments acting on the well due to longitudinal force, water- current, seismic effect etc. The earth below the maximum scour level is only effective in offering the passive resistance. Where greater external moments are required to be resisted by the passive earth pressure, greater grip length below the maximum scour level is required and therefore, to achieve this, further sinking of the well is necessary.

21.5.9 Design Considerations

21.5.9.1 As already stated in the previous para, the external moments acting on the wells due to various horizontal forces and the eccentric direct load are resisted by the moment due to passive earth pressure partly or fully depending on the magnitude of the passive pressure available which again is related to the area and nature of soil offering the passive resistance. The balance external moment, if there be any, comes to the base. The foundation pressure at the base of the well may, therefore, be calculated by the formula

$$f_b = \frac{W}{A} \pm \frac{M}{Z} \tag{21.19}$$

Where,

W	=	Total vertical direct load at the base of well after due consideration of the skin friction on the sides of wells.
A	=	Base area of well.
M	=	Moment at base.
Z	=	Section modulus of base.

21.5.9.2 The foundation pressure will be maximum when both W and M are maximum. This condition is reached when the live load reaction on the pier is maximum and no buoyancy acts on the well and the pier. On the other hand, the minimum foundation pressure and the possibility of tension or uplift may be expected when the live load reaction is minimum and full buoyancy acts due to which the dead weight of pier and well is reduced. The foundation pressure should be such that it remains within the permissible bearing power of the soil.

21.5.9.3 The skin friction acting on the sides of the wells is taken into account in balancing part of the direct load.

21.5.9.4 In estimating the steining thickness, it is necessary to find out the maximum moment as well as the maximum and minimum direct load on the steining. The steining thickness should be such that both the maximum and minimum stresses remain within the permissible value. In getting the maximum and minimum stresses, the considerations made in case of foundation pressure as outlined above should be tried here also. The steining stresses are obtained by using the following formula

$$f_s = \frac{W}{A} \pm \frac{M}{Z}$$

Where,

W	=	Total vertical load on the steining section under consideration.
A	=	Area of steining.
M	=	Moment at the steining section.
Z	=	Section modulus of the steining section.

21.5.9.5 The stability of well foundations shall be checked taking into account of all possible loading combinations including buoyancy or no buoyancy condition. Foundations for pier wells in cohensionless soil shall be designed on the basis of the *"Recommendations for Estimating the Resistance of Soils below the Maximum Scour level in the Design of Well Foundations of Bridges-IRC : 45-1972"*. Design of abutment wells in all types of soils and pier wells in cohesive soils shall be done in accordance with the recommendations in Appendix 4 of *"IRC : 78-1983-IRC Bridge Code — Section VII — Foundations and Substructure"*. Method of checking the stability of wells in predominantly clayey soil is explained below following the recommendations of IRC : 78-1983.

21.5.9.6 The active and passive earth pressure at any depth Z below the maximum scour level for a mixed type soil is given by :

$$6_a = \frac{\gamma Z}{N_\Phi} - \frac{2C}{\sqrt{N_\Phi}} \tag{21.20}$$

$$6_p = \gamma \cdot Z \cdot N_\Phi + 2C\sqrt{N_\Phi} \tag{21.21}$$

Where,

6_a	=	Active earth pressure per unit area of vertical section.
6_p	=	Passive earth pressure per unit area of vertical section.

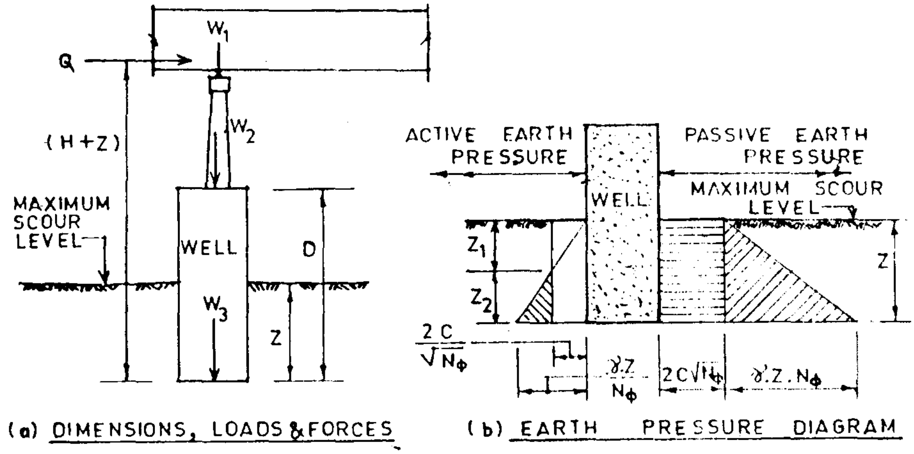


FIG. 21·22 STABILITY OF WELLS IN COHESIVE SOILS

C = Cohesion per unit area

N_Φ = $\tan^2\left(45° + \frac{\Phi}{2}\right)$

Φ = Angle of internal friction

γ = Unit weight of soil (submerged weight when under water).

For purely cohesive soils with $\Phi = 0$, equations 21.20 and 21.21 reduce to

$$6_a = \gamma Z - 2C \tag{21.22}$$

$$6_p = \gamma Z + 2C \tag{21.23}$$

21.5.9.7 Fig. 21.22(a) shows a well subjected to vertical concentric load W (= $W_1 + W_2 + W_3$) and a horizontal force Q acting at a distance H from maximum scour level. Fig. 21.22(b) shows the active and passive pressure diagrams based on equations 21.20 and 21.21 and also considering rotation at the base as recommended in Appendix 4 of IRC : 78-1983.

In Fig. 21.22(b) at a depth Z_1, $6_a = 0$

Hence from equation 21.20, $\frac{\gamma \cdot Z_1}{N_\Phi} - \frac{2C}{\sqrt{N_\Phi}} = 0$

$$\therefore \quad Z_1 = \frac{2C\sqrt{N_\Phi}}{\gamma} \tag{21.24}$$

If D is the external diameter of the well, total active pressure, P_a and passive pressure P_p acting on the well are given by :

$$P_a = \frac{1}{2}\left[\frac{\gamma Z}{N_\Phi} - \frac{2C}{\sqrt{N_\Phi}}\right] \times Z_2 \cdot D \tag{21.25}$$

$$P_P = \left[\frac{1}{2}\gamma \cdot Z \cdot N_\Phi \cdot Z + 2C\sqrt{N_\Phi} \cdot Z\right] D$$

$$= \left[\frac{1}{2}\gamma D Z^2 \cdot N_\Phi + 2C \cdot ZD\sqrt{N_\Phi}\right] \tag{21.26}$$

Moment at the base of well due to external horizontal force, $Q = Q\,(H + Z)$ (21.27)

Relief of moment at the base of well due to active and passive pressure of earth from equations 21.25 and 21.26

$$= M_p - M_a$$

$$= (P_p \times CG \text{ distance from base} - P_a \times CG \text{ distance from base})$$

$$= \left[\frac{1}{2}\gamma DZ^2 N_\Phi \times \frac{1}{3}Z + 2\cdot C\cdot Z\cdot D\sqrt{N_\Phi}\times\frac{1}{2}\cdot Z\right] - \left[\frac{1}{2}\left(\frac{\gamma Z}{N_\Phi} - \frac{2C}{\sqrt{N_\Phi}}\right)Z_2\cdot D\times\frac{1}{3}Z_2\right]$$

$$= \left[\left(\frac{1}{6}\gamma DZ^3 N_\Phi + CDZ^2\sqrt{N_\Phi}\right) - \frac{1}{6}\left(\frac{\gamma Z}{N_\Phi} - \frac{2C}{\sqrt{N_\Phi}}\right)D\cdot Z_2^2\right] \tag{21.28}$$

Equation 21.28 gives the ultimate nett moment of passive earth pressure. To arrive at the allowable moment of passive earth pressure from the ultimate moment ($M_p - M_a$) as given in equation 21.28, a factor of safety as given below shall be applied i.e., Allowable moment of passive resistance $= \dfrac{(M_p - M_a)}{F.O.S.}$

FOS for cohesive soil for load combination excluding wind or seismic forces shall be 3.0 and for load combination including wind or seismic shall be 2.4. The method of estimating base pressure of a well foundation is illustrated by the following example.

ILLUSTRATIVE EXAMPLE 21.6

Calculate the foundation pressures at the base of the circular well with the following particulars :

(a) Depth of well = 25.0 m

(b) Dia of Well = 8.0 m

(c) Depth below max. scour = 12.0 m

(d) Q = 100 t. acting at 37.0 m above the base of well under seismic condition.

(e) W_1 = Weight of Superstructure = 850 tonnes.

(f) W_2 = Weight of Pier = 150 tonnes.

(g) W_3 = Weight of Well = 900 tonnes.

(h) Soil around the well is mixed type having (i) $C = 0.2$ kg/cm² (ii) $\Phi = 15°$ (iii) γ (dry) = 1,800 kg/m³

(i) Permissible foundation pressures under seismic condition are 50 tonnes/m² and no tension.

Solution

(a) *Direct load at base of well*

(i) Load from superstructure	=	850 tonnes.
(ii) Weight of pier	=	150 tonnes
(iii) Weight of well	=	900 tonnes
		1,900 tonnes

(b) *Relief of dead load due to skin friction on well surface*

Skin friction = $C + K\,P_{Di}\tan\delta$

$P_{Di} = \gamma \times \dfrac{Z}{2} = 0.8 \times 6.0 = 4.8$ t/m²; K (at rest) = 0.6

$\delta = \Phi = 15°$, $\tan\delta = 0.2679$

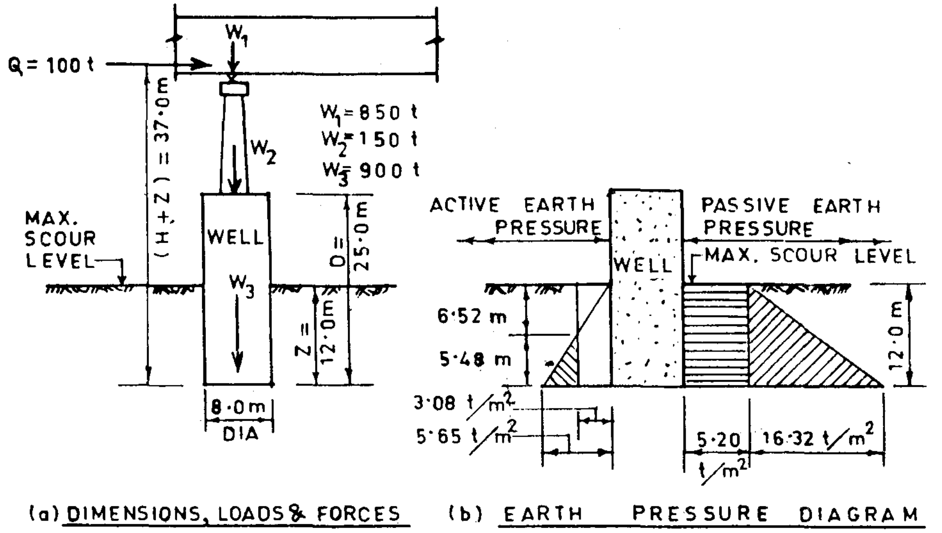


FIG. 21·23 (Example 21·6)

$\therefore$ Skin friction = $(2.0 + 0.6 \times 4.8 \times 0.2679) = 2.77$ t/m^2

$\therefore$ Total skin friction on the well surface below scour level = $\pi \times 8.0 \times 12.0 \times 2.77 = 835$ t

Taking a factor of safety of 2.0, skin friction available = $\dfrac{835}{2}$ = 418 tonnes.

Nett weight of well reduced by skin friction = 900 – 418 = 482 tonnes.

Buoyancy effect on well is not considered to get maximum foundation pressure as a worst case.

$\therefore$ Nett. D.L at well base = 850 + 150 + 482 = 1,482 tonnes.

(c) ***Moment at well base due to external horizontal force Q***

Moment = 100×37.00 = 3,700 tm.

(d) ***Relief of moment due to passive earth pressure on well***

$$N_\Phi = \tan^2\left(45^\circ + \frac{\Phi}{2}\right) = \tan^2\left(45^\circ + \frac{15}{2}\right) = 1.7$$

γ (submerged weight) = (1800 – 1000) = 800 kg/m^3 = 0.8 t/m^3

C = 0.2 kg/cm^2 = 2.0 tonnes/m^2

From equation 21.24, $Z_1 = \dfrac{2C\sqrt{N_\Phi}}{\gamma} = \dfrac{2 \times 2.0\sqrt{1.7}}{0.8} = 6.52$ m.

$\therefore$ $Z_2 = 12.0 - 6.52 = 5.48$ m ; $\dfrac{\gamma Z}{N_\Phi} = \dfrac{0.8 \times 12.0}{1.7} = 5.65$ t/m^2

$\dfrac{2C}{\sqrt{N_\Phi}} = \dfrac{2 \times 2.0}{\sqrt{1.7}} = 3.08$ t/m^2 ; $\gamma \cdot Z \cdot N_\Phi = 0.8 \times 12.0 \times 1.7 = 16.32$ t/m^2

$2C\sqrt{N_\Phi} = 2 \times 2.0\sqrt{1.7} = 5.2$ t/m^2

From equation 21.25, $P_a = \frac{1}{2}(5.65 - 3.08) \times 5.48 \times 8.0 = 56.33$ tonnes.

From equation 21.26,

$$P_p = \left[\frac{1}{2} \times 16.32 \times 12.0 \times 8.0 + 5.2 \times 12.0 \times 8.0\right] = (783.36 + 499.2) \text{ tonnes.}$$

Relief of moment due to passive and active pressure

= ($P_p \times$ C.G. distance from base.of well – $P_a \times$ C.G. distance from base well)

$$= \left(783.36 \times \frac{12.0}{3} + 499.2 \times \frac{12.0}{2}\right) - 56.33 \times \frac{5.48}{3} = (3133 + 2995 - 103) = 6025 \text{ tm}$$

FOS for sandy and clayey soils under seismic condition are 1.6 and 2.4 respectively. For a mixed soil as in the Illustrative Example FOS may be taken as 2.0.

$\therefore$ Safe passive moment of resistance $= \frac{6025}{2.0} = 3012$ tm.

(e) ***Design moment at base of well***

The design moment at base of well = External moment – relief of moment due to passive resistance = 3700 – 3012 = 688 tm

(f) ***Properties of well base***

$$A = \frac{\pi D^2}{4} = \frac{\pi (8.0)^2}{4} = 50.27 \text{ m}^2; \quad Z = \frac{\pi D^3}{32} = \frac{\pi (8.0)^3}{32} = 50.27 \text{ m}^3$$

$\therefore$ Foundation pressure at base of well as per equation 21.19

$$= \frac{P}{A} \pm \frac{M}{Z} = \frac{1482}{50.27} \pm \frac{688}{50.27} = 29.48 \pm 13.69 = 43.17 \text{ t/m}^2 \text{ or } 15.79 \text{ t/m}^2$$

Hence safe, as no tension occurs and the maximum foundation pressure is less than the allowable foundation pressure of 50.0 tonnes/m^2

21.5.10 Thickness of Well-Steining

21.5.10.1 The thickness of well-steining should be such that it can withstand the stresses developed due to loads and moments during service of the bridge. These stresses may be calculated by the procedure given previously. It is often observed that though the steining thickness satisfies all the loading conditions during service but it presents difficulties during sinking of the well. In such cases, either the steining becomes too light to give any sinking effort without addition of kentledge over the steining or failure of the steining occurs during sinking operation.

21.5.10.2 *"Sinking effort"* may be defined as the weight of the steining including kentledge, if any, per unit area of well periphery offering skin friction by the surrounding soil.

Thus, for a circular well without any kentledge,

$$\text{Sinking effort} = \frac{2\pi r t w}{2\pi R} = \frac{w r t}{R} \qquad (21.29)$$

Where,

r = Radius of the centre line of the steining.

t = Steining thickness.
w = Unit weight of steining.
R = Outer radius of well steining.

21.5.10.3 Unless the sinking effort exceeds the skin friction offered per unit area of steining surface, the sinking of the wells is not possible and therefore, the steining thickness should be made such that by adding small amount of kentledge, if necessary, the required amount of skinking effort is available in sinking the wells. In order to make economy in the well steining, it is sometimes preferred by some designers to adopt thin steining thickness as per theoretical calculation just sufficient for taking design loads during service of the bridge but this economy or saving in the steining is more than compensated by the additional cost of loading and unloading of the kentledge, increased cost of establishment charges due to delay in sinking the wells etc. According to ***Salberg, a practical Railway Engineer,*** this sort of economy aimed at by reducing the steining thickness is a false economy. His advice is —

> "The really important factor in well design is the thickness of the steining. It is regrettable feature that in most design, the steining thickness is cut down to what the designer fondly imagines is something really cheap ; money is saved on paper and in the estimate in the reduction of considerable masonry but in actual work it is all thrown away in the increased cost of sinking. A well that is too light in itself has to be loaded and the cost and delay of a well that has to be loaded to be sunk is terrible. You have nothing permanent for all the money you have spent in loading and unloading a well. Put your money into the steining and you have good money well spent and a solider and heavier well under your pier for ever. The chances are that you will save money on the job as a whole, you will save time and labour both important features, particularly the former when it is remembered that the period during which well can be worked at is limited to the low level duration of the river".

21.5.10.4 Empirical formula governing the thickness of steining for circular wells as required from sinking considerations is given below. This formula may be applicable to double-D or dumb-bell shaped wells also if the individual pocket is assumed to be a circular well of equivalent diameter.

$$t = k \cdot d \cdot \sqrt{D} \qquad (21.30)$$

Where,

t = Thickness of well steining in metres
d = External diameter of circular well or dumb-bell well or smaller dimension of double-D well in metres
D = Depth of well in metres below L.W.L. or G.L. whichever is higher
K = A Constant the value of which is given in Table 21.3

TABLE 21.3 : VALUES OF K IN EQUATION 21.30 *(IRC)*[3]

Type of Well	Concrete Steining		Brick Steining	
	Sandy Soil	Clayey Soil	Sandy Soil	Clayey Soil
Single circular or dumb-bell shaped	0.030	0.033	0.047	0.052
Double-D	0.039	0.043	0.062	0.068

Note 1 : *For boulder strata or for wells resting on rock where blasting may be required, higher thickness of steining may be adopted.*

Note 2 : *For wells passing through very soft clayey strata, the steining thickness may be reduced based on local experience.*

21.5.11 Sinking of Wells

21.5.11.1 The principal features in the sinking of wells are —

(a) To prepare the ground for laying the cutting edge.

(b) To cast the well-curb after laying the cutting edge.

(c) To build the steining over the well-curb.

(d) To remove the earth from the well pocket by manual labour or by grabbing and thus to create a sump below the cutting edge level. The well will go down slowly.

(e) To continue the process of building up the steining and the dredging in alternate stages. Thus the well sinks till the final founding level is reached.

(f) If necessary, kentledge load may be placed on the well steining to increase the sinking effort for easy sinking of the wells.

21.5.11.2 In preparing the ground for the cutting edge, it is not a problem when the location of the well is on a land or on a dry river bed but when the well is to be sited on the river bed with some depth of water, some special arrangements are to be made for laying the cutting edge depending upon the depth of water. These are :

(a) Open islanding.

(b) Islanding with bullah cofferdam.

(c) Islanding with sheet-pile cofferdam.

(d) Floating caisson.

(a) ***Open Islanding*** *(Fig. 21.24-a)* : When the depth of water is small say 1.0 m to 1.2 m, earth is dumped and an island is made such that its finished level remains at about 0.6 m to 1.0 m higher than the W.L. and sufficient working space (say 1.5 m to 3.0 m) round the cutting edge is available.

(b). ***Bullah Cofferdam*** *(Fig. 21.24-b)* : When the depth of water exceeds 1.2 m but remains within 2.0 m to 2.5 m, cofferdam is made by driving close salbullah piles and after placing one or two layers of durma mat, the inside is filled with sand or sandy earth. Sometimes, two rows of bullah piles at a distance of about 0.6 m between the rows are used and the annular space is filled with puddle clay. The unity of the inside and the outside rows being tied together gives more rigidity. This sort of islanding is adopted in comparatively deep water.

(c) ***Sheet Pile Cofferdam*** *(Fig. 21.24-c)* : Islanding with sheet pile cofferdam is resorted to when wells are sited inside river where the depth of water is considerable and bullah pile cofferdams are unsuitable for resisting the pressure of the filled up earth inside the cofferdam. The sheet pile cofferdams are stiffened with circular ring stiffeners.

(d) ***Floating Caissons*** *(Fig. 21.24-d)* : In very deep water, the sheet pile cofferdam is not a solution because the hoop tension developed due to the earth pressure of the filling material is tremendous. In such cases, floating caissons are usually employed. The well curb and the steining are made up to certain height with steel sheets braced inside with proper bracings. The space between the inside and outside surface is kept void. The caisson is floated and brought to the actual location. The *"launching"* of the caisson is done by filling the annular void space with concrete in stages. Before concrete filling, the caisson is carefully centered at its correct position. Due to the weight of the filled up concrete, the caisson goes down slowly and ultimately it touches the bed and it is grounded. The sinking is done as usual by building steining over the caisson and dredging. The grounding of the caisson in correct position sometimes may not be possible specially in high velocity rivers. In such cases, the caissons are refloated by pumping the water kept either in some cells of the multi-cell wells or in water tanks over the caissons and then re-grounded in correct position.

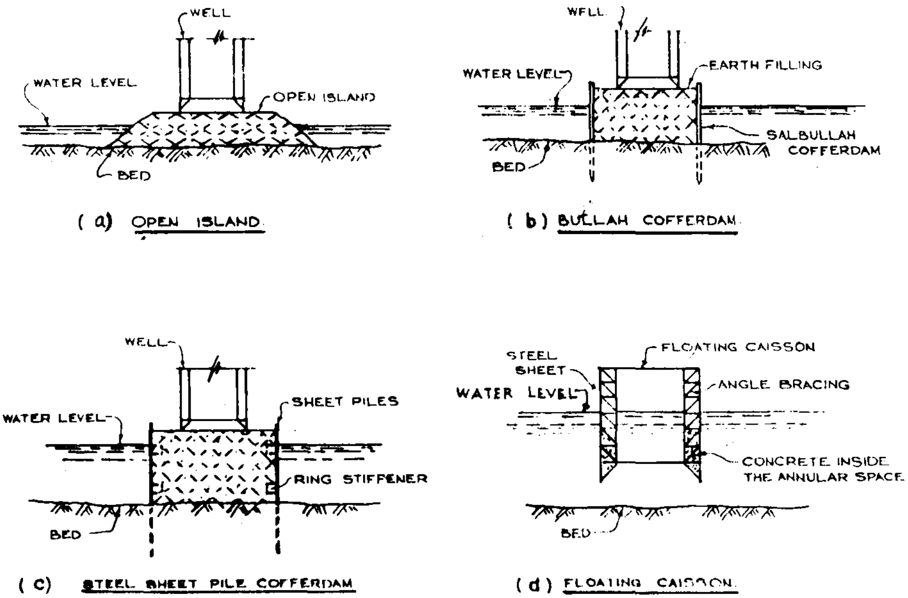


FIG. 21·24 VARIOUS METHODS OF STARTING WELL FOUNDATIONS

21.5.12 Method of Sinking

21.5.12.1 *Open Sinking* : Wells may be sunk by the open sinking (Fig. 21.25-a) or the pneumatic sinking method (Fig. 21.25-b). In the former method, the earth, sand, loose gravels etc. are removed from the bottom level of the cutting edge by means of grabbing or dredging and the well goes down due to its own weight. If the steining is lighter or if the skin-friction round the periphery of the well steining is greater, additional knetledge load may have to be applied to facilitate the sinking. Air-jetting near the cutting edge or water-jetting on the outside of the well-curb is resorted to when the well is stuck to a layer of stiff clay and it is found extremely difficult to sink the well further in spite of creating a deep sump under the cutting edge or placing a heavy kentledge on the well. If the jet-pipes are laid in sections as shown in Fig. 21.26(b) with one 100 mm diameter vertical pipe connected to 3 nos. 50 mm dia jet-pipes through a 100 mm dia horizontal pipe, these also help in rectifying the tilt since any one section situated on the high side can be utilised to loosen the friction on that side. Alternate chiselling and dredging yield results in sinking wells in hard strata.

Sometimes, the wells are partially dewatered to loosen the skin friction or to puncture the stiff layer of clay but it may be remembered that dewatering of the well is a very risky process since the well may sink suddenly which may lead to the heavy tilts and shifts or may cause cracks in the steining. Therefore, dewatering of the wells should not normally be attempted unless forced by circumstances. If dewatering is to be done at all, it should be done very slowly and carefully to avoid any awkward situation.

21.5.12.2 *Pneumatic Sinking* : Where open well sinking is likely to face many difficulties such as the presence of very hard stratum, loose boulders, inclined rock etc. or where the well is to be sunk some distance into rock, pneumatic sinking is adopted. In this method, a steel or a concrete air-lock is used at the bottom of the well. Compressed air pumped inside the air-lock displaces the water and workmen can work inside the air-lock without any difficulty. Two separate locks known as the man-lock and the muck-lock are provided at the top of wells. These are connected to the air-lock at bottom by means of an air-shaft and the work-men, tools and plant and the excavated materials are taken in or out through these man-lock or the muck-lock.

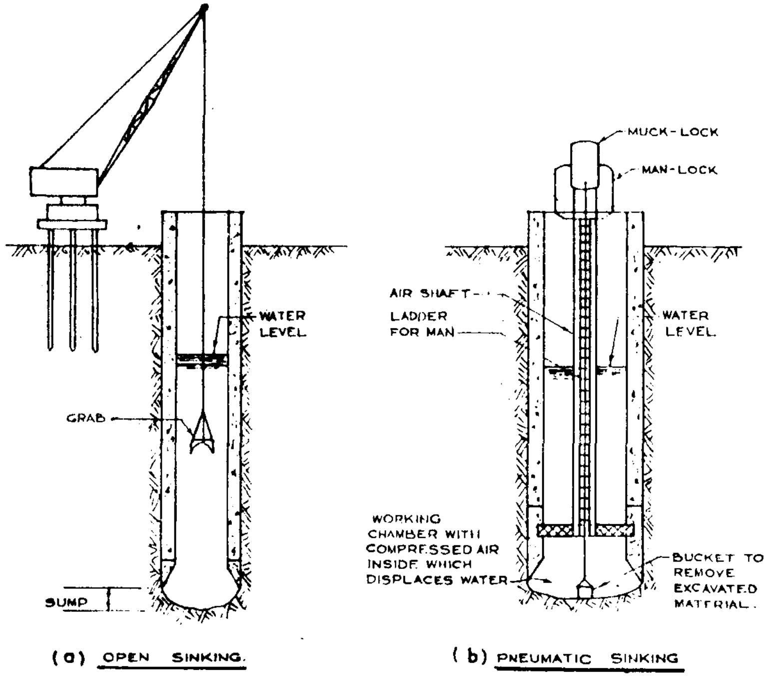


FIG. 21·25 METHOD OF SINKING

Provision for the installation of the pneumatic sinking should be made in cases where open sinking may normally serve the purposes but the possibility of sinking hazards are there and the pneumatic sinking may have to be resorted to. Normally, pneumatic sinking is more costly than the open sinking. The ratio of the cost depends on the difficulty or otherwise of the open sinking method. It is roughly estimated that pneumatic

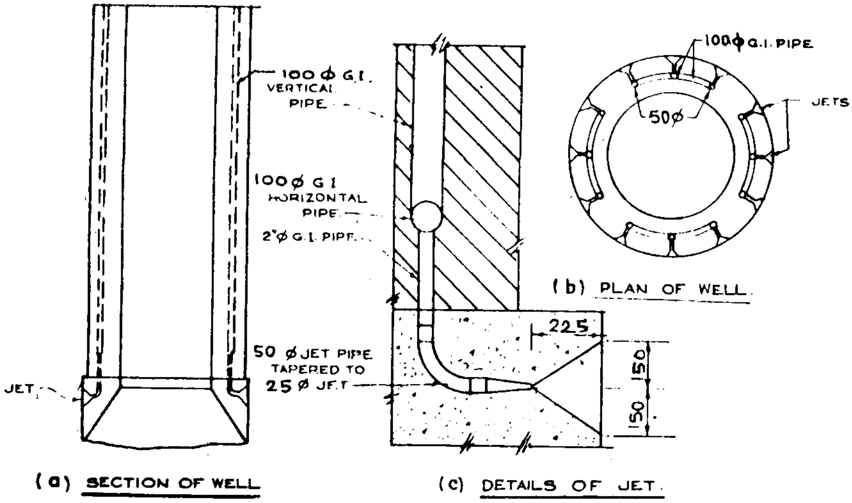


FIG. 21·26 WATER JETTING IN WELLS

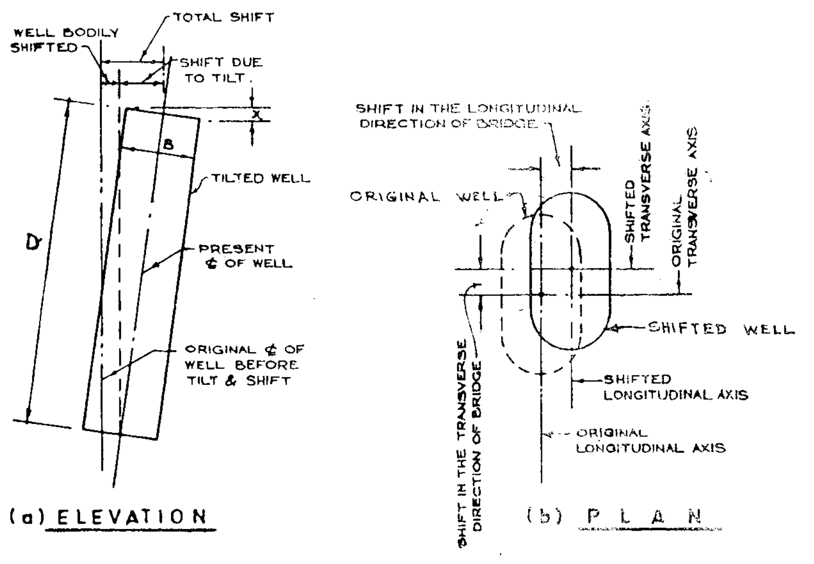


FIG.21·27 TILT AND SHIFT OF WELLS

sinking is two times expensive than the open sinking when the sinking conditions of the latter one are very favourable or moderately favourable. The former one may even be cheaper when the sinking by the latter method may have to face too many difficulties and the work is to be continued for a longer period under most adverse conditions.

21.5.13 Tilts and Shifts

21.5.13.1 The strata through which the wells are sunk are very rarely uniform and therefore, the resistance offered by these layers to the sinking is different in the different parts of the wells due to which tilt in the wells occurs. Sometimes, the thrust on the wells due to earth pressure vary in magnitude resulting in the shifting of the wells in some direction from the original position. The effect of the tilt on the well is to cause extra foundation pressure whereas the effect of the shift is to change the location of the pier. The shift of the well in the longitudinal direction causes change in the span lengths and the shift in the transverse direction causes the shifting of the centre line of the bridge. If the pier position is not shifted then the shift of the well also induces extra foundation pressure due to the eccentricity of the resultant vertical load on the wells. To counter-act the effect of tilt, it is always advisable to shift the pier on the high side so that the resultant direct load passes through the C.G. of the base area as far as possible. (Refer Illustrative Example 21.7)

21.5.13.2 Tilt is measured by taking level on the top of the steining or preferably on the gauge mark between the high side and the low side. If the difference of level between the high side and the low side is x (vide Fig. 21.27-a) and the distance between these two points is B then the tilt of the well is 1 in B/x. Generally, the allowable limit for tilt is 1 in 80. Allowable shift in any direction is 150 mm. In sinking wells through clayey soils, it is very difficult to keep the tilt within the aforesaid limit of 1 in 80 and higher tilts have to be accepted from practical considerations after due modification of the designs accordingly.

21.5.13.3 To rectify the tilt (and consequential shift), the following corrective measures are generally taken :

(i) To dredge near the cutting edge on the higher side if required after chiselling. Alternate chiselling and dredging generally yield results.

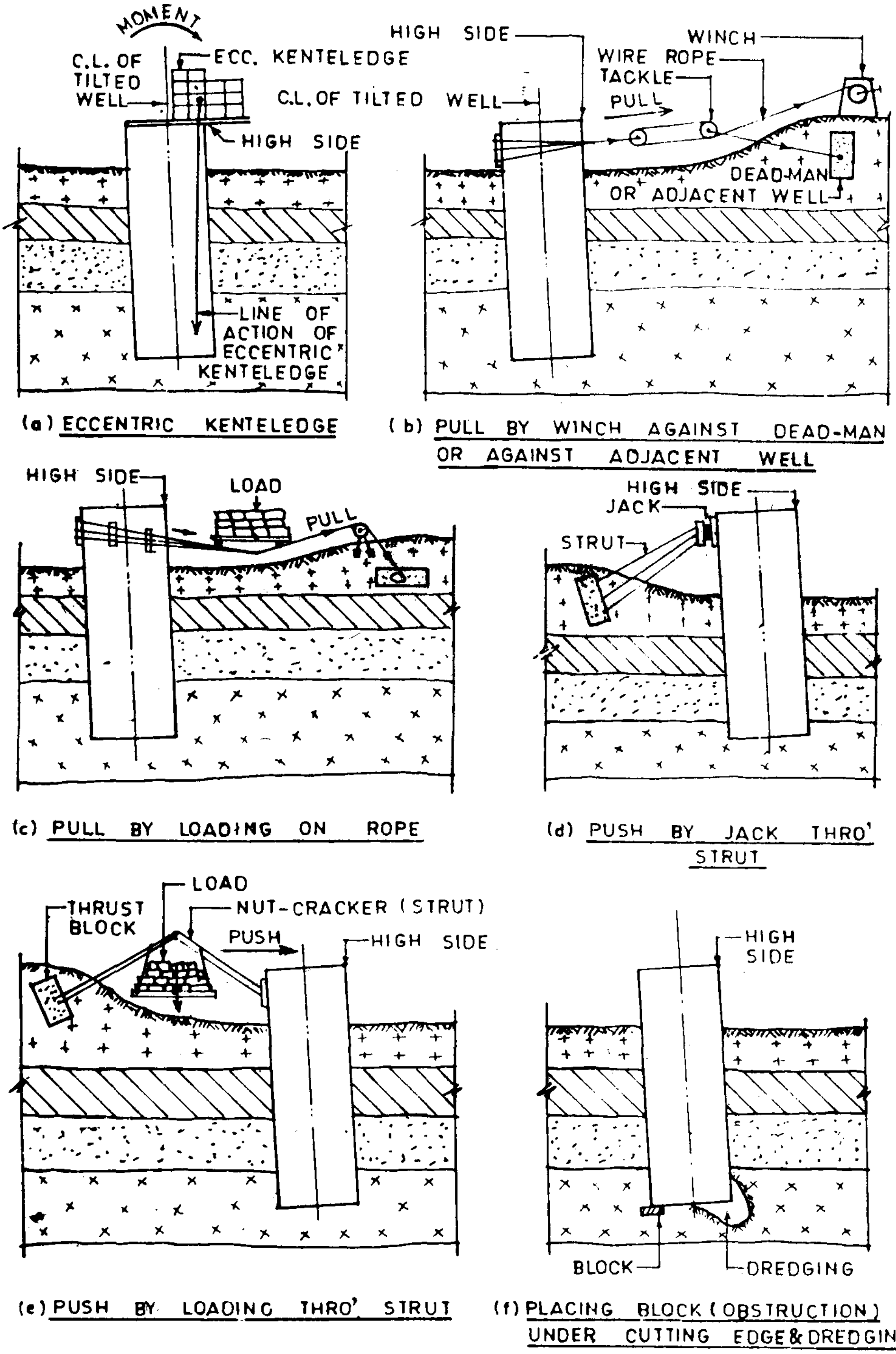


FIG. 21·28 VARIOUS METHODS FOR RECTIFICATION OF TILTS IN WELLS (Salberg)[18]

(ii) To apply air jetting or water-jetting on the outer high side so as to reduce the skin friction (Art. 21.5.12.1 and Fig. 21.26).

(iii) To apply eccentric kentledge (with positive eccentricity with respect to the base of well) on the high side (Fig. 21.28- a).

(iv) To pull the well at top on the high side (Fig. 21.28-b and 21.28-c).

(v) To push the well at top on the low side (Fig. 21.28-d and 21.28-e).

(vi) To place blocks or obstacles under the cutting edge on the low side and continue dredging on the high side below cutting edge (Fig. 21.28-f).

21.5.13.4 If in spite of adopting the above corrective measures, the tilt cannot be rectified to the permissible limits and if the actual foundation pressure exceeds the permissible value, it is not safe to plug the wells at the design foundation level as originally contemplated and as such the wells shall be sunk further down in order to get more relief due to passive pressure of earth and thus to bring the actual foundation pressure including the additional foundation pressure due to tilt and shift within the permissible limits. Deeper sinking will normally increase the allowable foundation pressure.

ILLUSTRATIVE EXAMPLE 21.7

If the well in Illustrative Example 21.6 is subjected to a final tilt of 1 in 50 and a true shift (in addition to shift due to tilt) of 0.3 m in the longitudinal direction, as shown in Fig. 21.29 (a), calculate the extra and total foundation pressures at the base of the well. How much shifting of the pier on the high side as recommended in Art. 21.5.13.1 is necessary to keep the foundation pressures within the allowable limits ?

Solution

From previous Illustrative Example 21.6 :

Weight of superstructure = 850 tonnes ; Weight of pier = 150 tonnes

Weight of well after allowing for skin friction = 482 tonnes

Depth of well = 25.0 m ; Z of well base = 50.27 m^3

Max. foundation pressure attained = 43.17 t/m^2 ; Allowable foundation pressure = 50.0 t/m^2

Due to a tilt of 1 in 50, the shift of well base = $\frac{25.0}{50} = 0.5$ m

From Fig. 21.29(a), it may be noted that due to the effect of tilt and actual shift the load from pier has an eccentricity of (0.5 + 0.3) = 0.8 m and the self weight of well acting at its C.G. i.e., 12.5 m above base has an eccentricity of $\frac{12.5}{50} = 0.25$ m.

Additional moment at well base due to tilt and shift = $(850 + 150) \times 0.8 + 482 \times 0.25$
= 800 + 120.5 = 920.5 tm.

Hence extra foundation pressure = $\frac{M}{Z} = \frac{920.5}{50.27} = 18.31\ t/m^2$

Hence total foundation pressure = $43.17 + 18.31 = 61.48\ t/m^2 > 50.0\ t/m^2$

To bring down the foundation pressure within the allowable limit, it is proposed to shift the well on the high side by 0.6 m as shown in Fig. 21.29 (b) thereby achieving a reduced eccentricity of 0.2 m for the load from pier, the eccentricity of self wt. of well remaining unchanged.

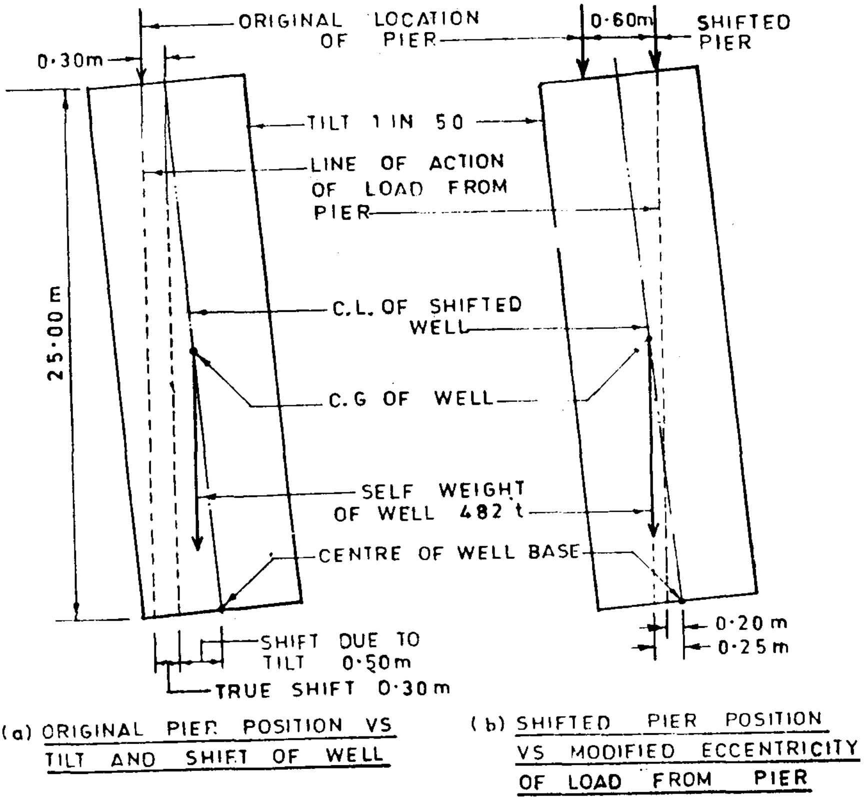


FIG. 21·29 (Example 21·7)

Hence reduced moment is M = (850 + 150) × 0.2 + 482 × 0.25 = 200 + 120.5 = 320.5 tm

Extra foundation pressure due to tilt and shift = $\frac{320.5}{50.27}$ = 6.38 t/m^2

Total foundation pressure = 43.17 + 6.38 = 49.55 t/m^2

This is within the permissible limit of 50.0 t/m^2. Hence safe. Thus, by shifting the pier by **0.6 m on the high** side of the well, reduction of moment due to tilt and shift is (850 + 150) × 0.6 = **600** tm which reduces the foundation pressure by $\frac{600}{50.27}$ i.e., 11.93 t/m^2 bringing down the excessive foundation pressure of 61.48 to (61.48 – 11.93) = 49.55 t/m^2 as obtained above. It is needless to mention that by shifting the pier as above, the original span arrangement is changed. The span on the left side increases by 0.6 m and the same on the right side reduces by 0.6 m.

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CHAPTER 22

BRIDGE BEARINGS, EXPANSION JOINTS AND WEARING COURSE

22.1 GENERAL

22.1.1 All bridges except arches and rigid frames should be provided with some sort of bearings depending on the spans whenever the span exceeds 8 metres. The necessity of using the bearings under the bridge supersructure is three-fold.

(i) Due to shrinkage and creep in concrete and temperature variation, contraction or expansion of the deck takes place and heavy secondary stresses may be developed if the above contraction or expansion of the deck is not allowed by providing free bearings at one end.

(ii) Under the action of load, the superstructure deflects and causes angular rotation at the supports. The bearings provided at the supporting end allow for this rotation otherwise heavy stresses may develop at the edges of the supports.

(iii) In addition, the bearings transfer the load from the superstructure to the supporting member uniformly over the full bearing area as far as possible.

22.1.2 Expansion joints are required to be provided at deck level at the junction between two adjacent spans or at the junction between the approaches and the end spans in order to facilitate free rotation and movement of the ends of the superstructure without causing any hindrance to the free flow of traffic.

22.1.3 The wearing course protects the structural concrete (specially the cover to the reinforcement) of the bridge deck from damage caused by the moving vehicles or by rain water. For efficient drainage, a camber or cross-fall of the bridge deck in the transverse direction is necessary which is easy to provide in the wearing course over the hardened deck.

22.2 TYPES OF BEARINGS

22.2.1 The bearings may be broadly classified into two groups depending on (a) the materials of which the bearings are made and (b) the specific functions they perform.

22.2.2 The bearings under the former group are :

(i) Mild Steel bearings

(ii) Stainless Steel bearings

(iii) Alloy bearings
(iv) High Tensile Steel bearings
(v) Cast Steel bearings
(vi) Elastomeric bearings
(vii) PTFE bearings
(viii) Reinforced Concrete bearings.

22.2.3 The bearings under the latter group are :

(i) Hinged bearings
(ii) Sliding bearings
(iii) Rocker bearings
(iv) Roller-cum-rocker bearing
(v) Pad bearings
(vi) Pot bearings

22.3 SELECTION OF BEARINGS

22.3.1 The type of bearing to be used in particular location should be judiciously selected since the bearing is a very critical component of the bridge structure and on it the economics of the bridge partially depends. The following considerations should be made while selecting the type of bearing to be used.

(a) ***Consideration of span length (simply supported) :***

(i) No bearings for slab bridges up to 8 m span except provision of roofing felt or tar paper.
(ii) Plate bearings or PTFE pad bearings beyond 8 m and up to 15 m span.
(iii) M. S. and R. C. rocker and roller-cum-rocker bearings beyond 15 m and up to 30 m span.
(iv) Neoprene pad bearings beyond 8 m and up to 30 m.
(v) Neoprene pot bearings and steel roller and roller-cum-rocker bearings beyond 30 m.

(b) ***Consideration of frictional resistance :*** The coefficient of friction for roller bearing is 0.03 and that for sliding plate bearing is 0.15 to 0.25 i.e., 5 to 8 times that of roller bearing. The longitudinal force coming over the piers or abutments depends on the vertical reaction and the frictional resistance of the free bearings. The design of foundations with long rigid type piers and abutments are greatly influenced by this force and therefore, if bearings with higher values of frictional resistance are used over long piers or abutments, the cost of the substructure and foundation is increased. Therefore, in the superstructure resting on long piers and abutments though of lesser span, bearings with less frictional resistance even at some extra cost may economise the substructure and foundation cost considerably. Table 5.5 (Chapter 5) gives the coefficient of frictional resistance of various types of bearings to be used in design.

22.4 MILD STEEL BEARINGS

22.4.1 Mild steel may be used in the manufacture of rocker, roller or plate bearings. To keep mild steel bearings free from rust, these are often immersed in grease by providing grease boxes (Fig. 22.15). However, it has been observed that due to lack of proper maintenance, mild steel bearings get rusted in course of time resulting in actual increase of the coefficient of friction over the design value. This generates additional horizontal force on the top of piers and abutments. For this reason and also due to the availability of other type of bearings suitable for this span range viz., noeprene, PTFE etc., mild steel plate bearings are not frequently used as was done previously.

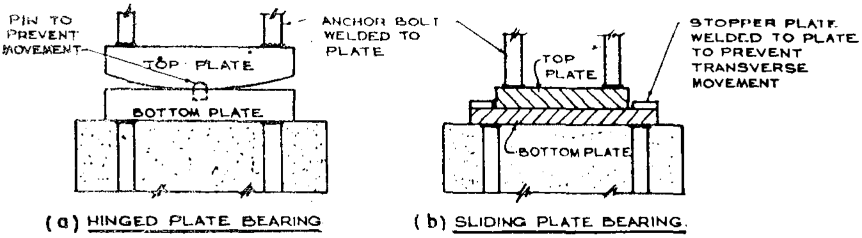


FIG. 22·1 PLATE BEARINGS

22.5 HARD COPPER ALLOY AND STAINLESS STEEL BEARINGS

22.5.1 Hard copper alloy or stainless steel plate bearings are favoured more than M. S. plate bearings because they are rust free as well as have less frictional resistance.

22.5.2 Similar to rocker and roller bearings, two types of plate bearings are used, viz. hinged plate bearing at the fixed end (Fig. 22.1-a) and the sliding plate bearing (Fig. 22.1-b) at the free end. The hinged bearing is composed of one curved top plate over a flat bottom plate with a pin at the centre which allow rotation but prevent transaltion in any direction. The sliding bearing consists of one plate over the other with graphite or grease between the plates for easy movement. Stopper plates are welded to the bottom plate to prevent sidewise movement.

22.6 STEEL BEARINGS

22.6.1 Steel Roller

22.6.1.1 Roller bearings permit both the linear and the rotational movements. Single roller (Fig. 22.2-a) is used for roller bearings of moderate capacity but when roller bearings are to be designed for larger capacities, the number of rollers is increased by keeping the diameter of the rollers nearly the same as for single roller bearing. Flawless casting of rollers having diameter more than 200 mm becomes difficult and in such cases detection of defects in castings such as tapped air, bubbles etc. by X-ray test becomes difficult if rollers are made of larger diameter. When the number of rollers in an assembly of rollers exceeds two, the permissible load on each roller is reduced.

22.6.1.2 In multiple roller bearings (Fig. 22.2-b), one intermediate plate known as *"Saddle Plate"* is inserted between the assembly of rollers and the top plate. The saddle plate functions as a medium for allowing both the rotation and the translation.

22.6.1.3 The rollers are prevented from excess rolling by providing lugs or stopper plates, the movement in the transverse direction being prevented by the guides. These guides also ensure uniform and regular movement of the rollers. The assembly of rollers is connected by a tie bar in order to maintain fixed spacing of the rollers during movement.

22.6.1.4 Pendulum shaped segmental rollers (load carrying capacity reduced by 50%) are sometimes made by eliminating the sides of the complete circle in order to save some materials but full circle rollers are preferable to segmental rollers since the former relieves the heavy stresses at the point of contact in a better way. Moreover, it has been observed that full circle roller bearings have prevented the superstructure from dislodging even when there was excessive tilt or rotation in the roller due to differential settlement of the foundation. Segmental rollers, if used in such cases, could not have averted the dissaster.

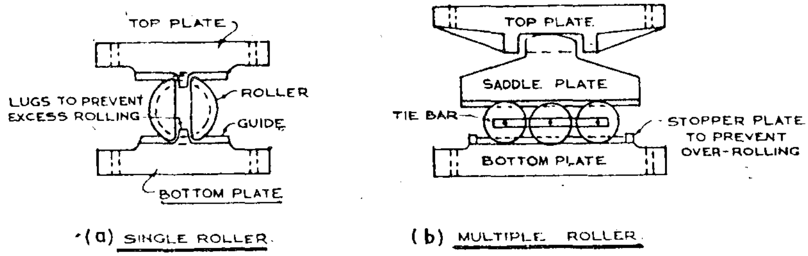


FIG. 22·2 STEEL ROLLER BEARINGS (Rakshit)[8]

22.6.2 Steel Rocker

22.6.2.1 While the roller bearings permit both the rotation and the translation of the ends of the superstructure, the rocker bearings permit only the rotation. The bearing is termed *"Rocker"* because the top plate rocks over the bottom plate.

22.6.2.2 In Fig. 22.3, two types of rocker bearing are shown. The difference in the types lies in the arrangement of preventing longitudinal and transverse movement of the top plate and also in the rocking surface — in one, flat surface rocks over convex surface and in the other, convex surface rocks over flat surface.

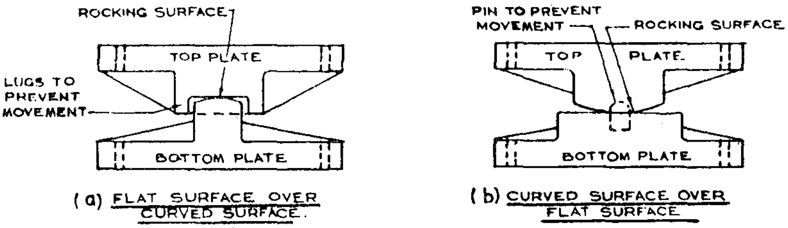


FIG. 22·3 STEEL ROCKER BEARINGS (Rakshit)[8]

22.7 ELASTOMERIC PAD BEARINGS

22.7.1 Elastomeric bearings may be made from either natural rubber or synthetic rubber. Neoprene pad bearings made from synthetic rubber are generally used in India. The vertical load from the superstructure is taken by the neoprene bearings when compressive strain and compressive stress are developed in the neoprene pad (Fig. 22.4-a). The horizontal force from the superstructure is, however, resisted by the shear strain and shear stress (Fig. 22.4-b). In case of rotation of the superstructure in the vertical plane due to load and other

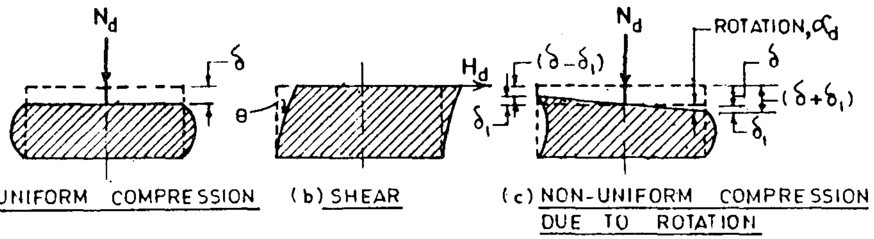


FIG. 22·4 STRAINS IN ELASTOMERIC BEARINGS DUE TO VARIOUS EFFECTS

effects, the uniform compressive strain produced by the vertical load is increased on one side and reduced on the other side (Fig. 22.4-c).

22.7.2 Un-strained neoprene pads bulge more (Fig. 22.4-a) thereby reducing their load carrying capacity and as such restrained neoprene bearings are used. In these restrained pads, steel or laminates are interposed between multi- layer pads as shown in Fig. 22.5-a. These steel laminates are well bonded by the process of vulcanisation with the neoprene layers and thus reduce the bulging effect and consequently increase their load carrying capacity (Fig. 22.5-c and 22.5-d).

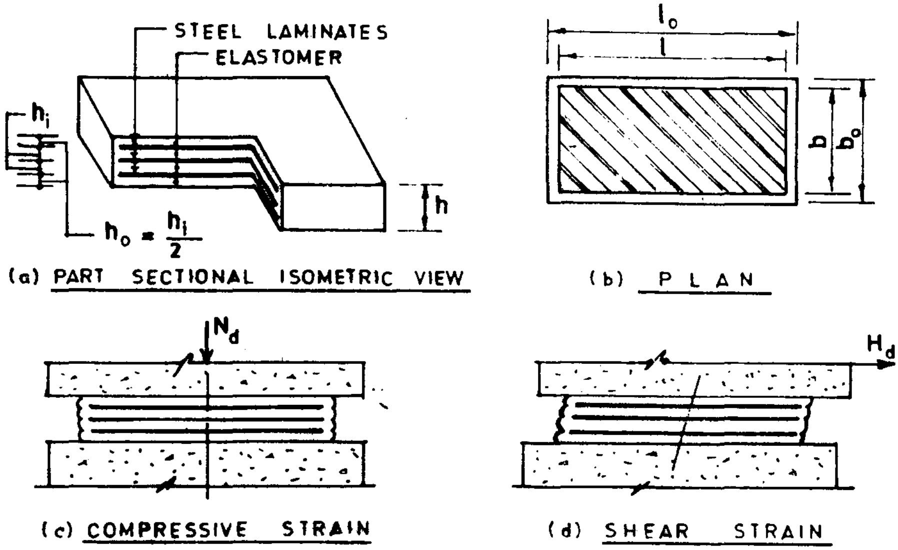


FIG. 22·5 DETAILS OF RESTRAINED ELASTOMERIC BEARING

22.8 ELASTOMERIC POT BEARINGS

22.8.1 As stated in Art. 22.3.1, elastomeric pad bearings can be used up to a span of 30 m approximately. When the span is more, both the vertical load and the rotation on the bearings are large and as such the pad bearings in such cases are found unsuitable. Pot bearings which are confined elastomeric bearings are the answer for such situation. The pot bearings are composed of relatively thin unreinforced circular neoprene pad completely enclosed in a steel pot having a circular enclosure for the neoprene pad (Fig. 22.6).

22.8.2 The teflon layer provided in between the top plate and the intermediate plate permits horizontal movement of the deck while the confined neoprene pad inside the pot allows the rotation. This type of bearings

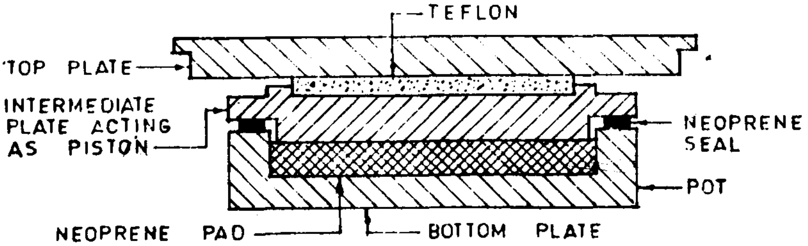


FIG. 22·6 POT BEARING (Victor)[15]

are ideally suitable for skew and curved bridges where the direction of movement varies and these bearings can take both the translational and rotational movement in any direction.

22.9 PTFE PAD BEARINGS

22.9.1 PTFE (Poly Tetra-Fluoro-Ethylene) is a thermoplast and available under various trade names such as Teflon, Hostaflon, TF, Algoflon and Fluon etc. The polymer has a great molecular strength, chemical inertness and low coefficient of friction.

22.9.2 Pure PTFE is not used in bridge bearings since it has low resistance to wear and susceptibility to cold flow or plowing tendency under compressive loads. Therefore, certain filler materials and reinforcing agents such as glass fibre, graphite molybdenum sulphide etc. or a combination of them are mixed. In the latter case, however, the low frictional properties are sacrificed to some extent.

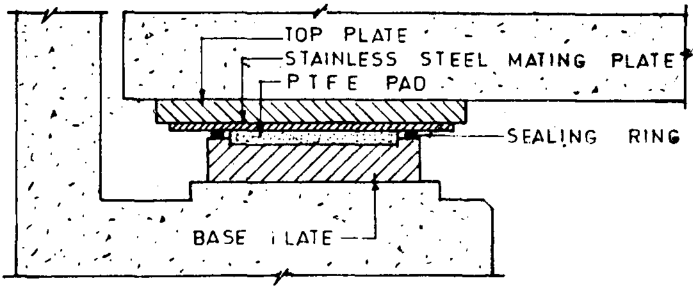


FIG. 22·7 PTFE PAD BEARING (Mahalaha)[5]

22.9.3 Two PTFE pads may be used, one sliding against the other but in such case there is possibility of more plowing (creep) effect specially under very high pressures. Therefore, it is usually placed below a matting plate, normally a stainless steel plate, which is corrosion and weather resistant. However, in such case the frictional properties are reduced to some extent than those available with two PTFE pads. It has been observed that even with a stainless steel matting plate, the micro thin film of PTFE gets transferred on to the matting plate after few movements and creats a condition as if the sliding takes place between two PTFE surfaces. The matting plate shall have sufficient margin on both sides beyond the PTFE pad so that even after the sliding, loads from the superstructure gets properly transferred to the PTFE pads.

22.9.4 The PTFE pads shall be properly bonded at the base with a base plate or a backing plate either a steel plate or a reinforced elastomeric pad with a view to eliminate or substantially minimise creep under loads. PTFE pads may be bonded with high temperature epoxy adhesives under factory controlled condition.

22.10 REINFORCED CONCRETE BEARINGS

22.10.1 Mild steel bearings suffer from the drawbacks as stated in Art. 22.4.1. Cast steel bearings are usually very costly and not easily available from the manufacturers and therefore, for medium span bridges where the use of roller and rocker bearings is obligatory, R. C. bearings are sometimes used. Since the bearings are most vulnerable part of a bridge structure, special care should be taken in the manufacture of such bearings. A rich concrete mix of 1 : 1 : 2 is usually specified for the manufacture of bearings.

22.10.2 R. C. Roller

R. C. rollers are reinforced heavily both in the direction of transmission of load as well as in the transverse direction to prevent the tendency of bursting of the roller (Fig. 22.8). Interlocking spirals are provided both vertically and horizontally for this purpose. Dowel bars with copper lining are provided from the pier or abutment cap up to the deck through the elliptical holes of the rollers. These holes permit rolling of the rollers when required whereas the dowel bars prevent over-rolling of the rollers. Lead sheet of 6 mm to 10 mm thickness is used both at the top and bottom of the rollers for uniform distribution of the load over the rollers. The lead sheet also helps in easy rolling of the rollers.

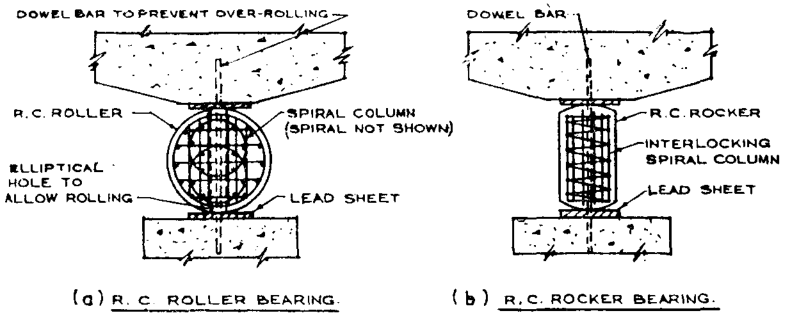


FIG. 22·8 REINFORCED CONCRETE BEARINGS (Rakshit)[8]

22.10.3 R. C. Rocker

R. C. Rocker is nothing but a R. C. segmental roller. Unlike cast steel rocker bearing, R. C. rocker bearing permits both the rotations and translation (though to some lesser extent) of the deck similar to roller bearings. But in case of rocker, no elliptical hole is provided and the dowel bar holds the deck in a semi-hinged condition. As already stated in case of cast steel bearings, in Art. 22.6.1.4, full circle bearings distribute loads in a better way than the segmental bearings. This is true for segmental R. C. roller and rocker also and as such these are not recommended for adoption. Full circle R. C. roller bearings as shown in Fig. 22.8 were used in West Bengal in a number of bridges. Instead of R. C. rocker (segmental roller), the type of rocker shown in Fig. 22.9 may be adopted.

22.10.4 Curved Pier Top (Rocker)

Sometimes the pier tops are made curved and the superstructure rests on it with a lead sheet in between them. The function of the lead sheet is to distribute the load evenly on the pier cap from the superstructure.

Dowel bars lined with 16 gauge copper sheet lining to prevent rusting are used to keep the deck in position. The curved top of the pier functions as a rocker bearing, and as such shall be provided with proper radius of curvature for adequate load transference. Dispersion grids or spirals as necessary shall be provided both in the superstructure and the pier cap as outined in Art. 22.11.5. This type of bearing is very economical and may be suitable for medium span superstructures.

22.11 DESIGN OF BEARINGS

22.11.1 Top and Bottom Plates of Roller and Rocker Bearings

The area of the top and bottom plates of roller, rocker or plate bearings may be determined from the load to be carried and the safe allowable pressure between concrete and steel surfaces. The permissible direct stress in concrete can be increased as given by equation 22.1 if dispersion grids are provided. However, if spiral columns are provided, greater value may be permitted as given in equation 22.12. The thickness of plates may be found out from shear or bending considerations.

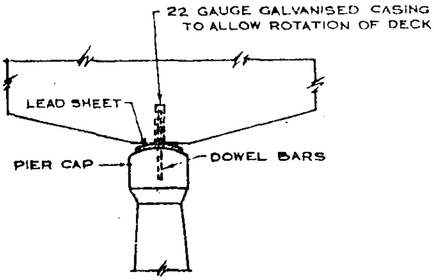


FIG. 22·9 CURVED PIER TOP ROCKER (Rakshit)[8]

22.11.2 The allowable bearing pressure, 6_a under a bearing shall be given by

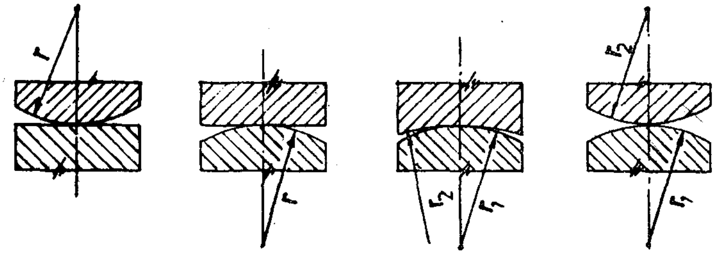
$$6_a = 6_{co}\sqrt{\frac{A_1}{A_2}} \qquad (22.1)$$

Where	6co	=	The permissible direct compressive stress in concrete at the bearing area of the base.
	A_1	=	Dispersed concentric area which is geometrically similar to the loaded area A_2 and also the largest area that can be contained in the plane of A_1. Maximum width of dispersion beyond the loaded area face shall be limited to twice the height.
	A_2	=	Loaded area, and
	$\frac{A_1}{A_2}$	$\not>$	2

22.11.3 Design of Roller or Rocker

The contact surface between a roller and the bottom plate is convex surface over flat surface (Fig. 22.10a) whereas the same between a top plate and the roller is flat surface over convex surface (Fig. 22.10b). The contact surface between the top or bottom plate and the rocking surface may be any of the following :

i) Convex surface over flat surface (Fig. 22.10a)
ii) Flat surface over convex surface. (Fig. 22.10b)
iii) Concave surface of larger radius over convex surface of smaller radius. (Fig. 22.10c).
iv) Convex surface over convex surface. (Fig. 22.10d)



(a) (b) (c) (d)

FIG. 22·10 VARIOUS TYPES OF BEARING SURFACES

In determining the radius of curvature of the contact surface of roller or rocker bearings, the general formula given by W. L. Scott in his book *"Reinforced Concrete Bridges"* is

$$\frac{1}{p}=\frac{1}{K}\left(\frac{1}{r_1}+\frac{1}{r_2}\right) \tag{22.2}$$

Where,

p = Pressure per inch length of the bearing in lbs.

K = A constant depending on the material used and is equal to 342 for concrete ($1:1\frac{1}{2}:3$), 1200 for mild steel and 2840 for cast steel.

r_1 and r_2 = Radii of curvature in inches. For convex surface +ve sign and for concave surface –ve sign.

For the radii of curvature shown both in Fig. 22.10a and 22.10b, equation 22.2 becomes,

$$\frac{1}{p}=\frac{1}{K}\left(\frac{1}{r}+\frac{1}{\alpha}\right)=\frac{1}{Kr} \tag{22.3}$$

For Fig. 22.10c,

$$\frac{1}{p}=\frac{1}{K}\left(\frac{1}{r_1}-\frac{1}{r_2}\right) \tag{22.4}$$

For Fig. 22.10d.

$$\frac{1}{p}=\frac{1}{K}\left(\frac{1}{r_1}+\frac{1}{r_2}\right) \tag{22.5}$$

If p is given in Newton per mm. length in place of pounds per inch length and if r_1 and r_2 are given in mm. in place of inch, the equation 22.3 for cast steel bearings with K = 2840 becomes,

$$\frac{1}{p}=\frac{(2.2\times25.4\times25.4)}{2840\times9.8}\times\frac{1}{r}=\frac{1}{19.6}\times\frac{1}{r}$$

$$\text{i.e., } p = 9.8\ d \tag{22.6}$$

Similarly, equation 22.4 becomes,

$$\frac{1}{p}=\frac{1}{19.6}\left(\frac{1}{r_1}-\frac{1}{r_2}\right)$$

Putting $r_1=\frac{d_1}{2}$ and $r_2=\frac{d_2}{2}$ the above equation becomes,

$$\frac{1}{p}=\frac{2}{19.6}\left(\frac{1}{d_1}-\frac{1}{d_2}\right)$$

$$\text{or } p=\frac{9.8}{\left[\frac{1}{d_1}-\frac{1}{d_2}\right]} \tag{22.7}$$

"IRC : 83-1982-Section IX-Metallic Bearings" gives the allowable loads on the cylindrical rollers based on the aforesaid principles with some modified values of the constants for mild steel and high tensile steel. These are reproduced below (p is given in N per mm. and d in mm.).

(a) ***Cylindrical Rollers on Flat Surfaces***

$$p = Kd \tag{22.8}$$

(b) *Cylindrical Rollers on Curved Surfaces*

$$p = \frac{K}{\left[\frac{1}{d_1} - \frac{1}{d_2}\right]} \qquad (22.9)$$

The values of K in equations 22.8 and 22.9 both for mild steel and high tensile steel and also for single or double rollers and three or more rollers are given in Table 22.1

TABLE 22.1 : VALUES OF K IN EQUATIONS 22.8 AND 22.9 *(IRC)*[1]

Nos. of rollers	Values of K	
	For Mild Steel and Cast Steel	For High Tensile Steel
Single and double	8	10
Three or more	5	7

For reinforced concrete rollers on flat surface, the value of K when p is in Newton per mm length and d is in mm is evaluated as before.

$$K = \frac{342 \times 9.8}{2.2 \times 25.4 \times 25.4} = 2.36 \text{ Say } 2.4$$

$$\therefore \frac{1}{p} = \frac{1}{2.4} \times \frac{1}{r} \text{ i.e. } p = 2.4\,r$$

$$\text{or } p = 1.2\,d \qquad (22.10)$$

For reinforced concrete rollers on curved surfaces.

$$p = \frac{1.2}{\left[\frac{1}{d_1} - \frac{1}{d_2}\right]} \qquad (22.11)$$

22.11.4 Design of Elastomeric Bearings

The design of elastomeric bearings requires the following values of local effects :—

i) Normal loads, Nd
ii) Horizontal loads, Hd
iii) Imposed translation, Hd
iv) Rotation, αd.

The bearings shall satisfy the limiting permissible values in respect of the following :

i) Translation
ii) Rotation
iii) Total shear stress due to axial compression, horizontal deformation and rotation
iv) Friction.

The design principles have been elaborated in *"IRC:83-(Part II) — 1987 — Section -IX — Elastomeric Bearings"* which may be referred for guidance.

22.11.5 Design of Dispersion Grids & Spirals

When the intensity of bearing pressure between the bearing plates and the concrete surface exceeds the allowable value, dispersion grids and spirals are provided to distribute the load on a wider area in order to bring down the pressure to within safe limits. Where the increase in the concrete stress beyond the allowable value is not significant, only dispersion grids may be used in two layers.

Dispersion grids are closely spaced reinforcement of 6 mm to 10 mm diameter with 50 mm to 75 mm pitch as shown in Fig. 22.14. Usually two layers of dispersion grids at 75 mm to 100 mm apart are placed above the top plate or below the bottom plate.

The spirals are composed of longitudinal bars tied with closely spaced binders in the form of helix. The spirals function as R. C. columns and transfer the load from the bearing to the concrete surface after proper dispersion so that the intensity of pressure coming on the concrete surface is within the safe value. When load is distributed over the concrete through the dispersion grid and the spiral column, the allowable concrete stress just behind the bearing plate may be increased beyond the value given by the formula in equation 22.1 which is applicable in cases where dispersion grids are only provided.

The load on the spiral column should not exceed the value given by :

$$P = A_C\, 6_{CO} + A_S\, 6_{SO} + 2A_{SP}\, 6_{SP} \tag{22.12}$$

Where

P	=	Load on spiral coltumn in Newton
6_{CO}	=	Permissible direct stress of concrete, in MPa
6_{SO}	=	Permissible stress for longitudinal steel in direct compression in Mpa.
A_C	=	Cross-sectional area of concrete in the column core (excluding the area of longitudinal steel) in mm^2
A_S	=	Cross-sectional area of longitudinal steel in mm^2.
6_{SP}	=	Permissible stress in tension in spiral reinforcement = 95 MPa
A_{SP}	=	Equivalent area of spiral reinforcement (i.e., the volume of spiral reinforcement per unit length of column).

In no case the sum of the terms $A_C\, 6_{CO}$ and 2 A_{SP}. 6_{SP} shall exceed 0.5 fck.

22.12 MATERIALS AND SPECIFICATIONS FOR BEARINGS

22.12.1 For materials and specifications for metallic bearings, clause 904 of *"IRC : 83-1982 — Section IX — Metallic Bearings"* and for elastomeric bearings, clause 915 of *"IRC : 83 (Part II) — 1987 — Section IX — Elastometric Bearings"* shall be referred.

22.12.2 Permissible stresses in steel used for metallic bearings are given in Table 22.2.

TABLE 22.2 : PERMISSIBLE STRESSES IN STEEL FOR BEARINGS *(IRC)*[1]

No.	Components of bearings	Permissible stress, MPa	
		Mild Steel and Cast Steel	High Tensile Steel
1	Parts in bending (tensile or compressive) on effective sectional area for extreme fibre stress —		
	(a) For plates, flats, round square and similar sections	160	200
	(b) For pin	205	295

TABLE 22.2 : PERMISSIBLE STRESSES IN STEEL FOR BEARINGS (IRC)[1] (Contd.)			
No.	Components of bearings	Permissible stress, MPa	
		Mild Steel and Cast Steel	High Tensile Steel
2	Parts in shear		
	(a) Maximum shear stress on plates	105	140
	(b) Max. shear stress for turned and fitted bolts and pins.	100	0.43 fy where fy is the yield stress.
	(c) Max. shear stress in black bolts and rocker pins	85	0.37 fy
3	Parts in bearings		
	(a) On flat surface	185	240
	(b) Knuckle pin and black bolts	200	0.87 fy

ILLUSTRATIVE EXAMPLE 22.1

Design a mild steel roller bearing for a load of 1000 KN inclusive of impact effect. Given :

- ***i) Coefficient of friction of roller bearing = 0.03 and***
- ***ii) Movement of roller on either direction = 20 mm***

Solution

Design of top and bottom plate

Assume length of plate as 650 mm and permissible base pressure (direct compression) as 4.5 MPa

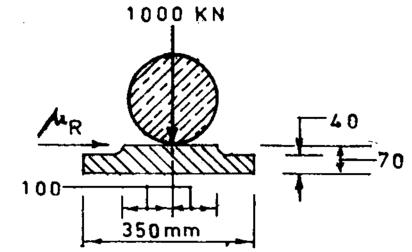


FIG. 22·11 (Example 22·1)

$\therefore$ Width of plate $= \dfrac{1000 \times 10^3}{650 \times 4.5} = 342$ mm.

Adopt a plate size of 650 × 350 mm.

$\therefore$ Base pressure $= \dfrac{1000 \times 10^3}{650 \times 350} = 4.4$ MPa

Moment due to eccentric loading for the translation of bearing $1000 \times 20 = 20{,}000$ KN mm.

Moment due to frictional resistance of bearing (considering thickness of plate as 70 mm)

$= 0.3 \times 1000 \times 70 = 2100$ KN mm

Total moment = 20,000 + 2,100 = 22,100 KN mm

Section modulus of base plate $= \dfrac{650 \times 350^2}{6} = 13{,}270 \times 10^3$ mm. units

Additional base pressure due to moment $= \dfrac{M}{Z} = \dfrac{22{,}100 \times 10^3}{13{,}270 \times 10^3} = 1.67$ MPa

Total base pressure = 4.4 + 1.67 = 6.07 MPa

Basic permissible concrete stress in compression, 6_{co}, from Table 5.9 for M20 concrete = 5.0 MPa

Increased permissible value may be obtained from equation 22.1 by using dispersion grids.

Assuming a pedestal size of 750 × 450 × 150 mm,

$A_1 = 750 \times 450$ and $A_2 = 650 \times 350$

$$\therefore\ 6_a = 6_{co}\sqrt{\frac{A_1}{A_2}}$$

$$= 5.0\sqrt{\frac{750 \times 450}{650 \times 350}} = 6.09 \text{ MPa}$$

Actual base pressure = 6.07 MPa. Hence safe. Considering one mm. length of plate,

$M_{xx} = 4.4 \times 75 \times 37.5 + \frac{1}{2} \times 0.72 \times 75 \times \frac{2}{3} \times 75$

$= 12{,}375 + 1350 = 13{,}725$ Nmm

Allowable stress in steel in bending from Table 22.2 = 160 MPa

$\therefore$ Z (required) $= \frac{13{,}725}{160} = 85.78$ mm. units

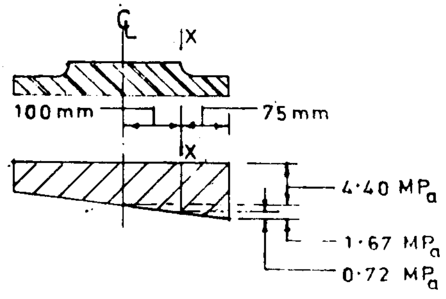


FIG. 22·12 (Example 22·1)

If t be the thickness of the plate at XX, $Z = \frac{t^2}{6}$

$\therefore\ \frac{t^2}{6} = 85.78$ or $t = \sqrt{85.78 \times 6} = 22.7$ mm.

Make t = 35 mm.

Moment under the eccentric load

$= 4.4 \times 155 \times \frac{1}{2} \times 155 + \frac{1}{2} \times 1.67 \times 155 \times \frac{2}{3} \times 155$

$= 52{,}855 + 13{,}370 = 66{,}225$ Nmm.

Moment at centre when the load is concentric = $4.4 \times 175 \times \frac{1}{2} \times 175 = 67{,}375$ Nmm.

$\therefore$ Z (required) at centre $= \frac{67{,}375}{160} = 421$ mm units

$\therefore\ \frac{t^2}{6} = 421$ or $t = \sqrt{421 \times 6} = 50.26$ mm.

Make t at mid section = 70 mm.

Design of Roller

Overall length of roller = 650 mm

Effective length of roller after deduciton for guide, end lug etc. = 650 – 50 (say) = 600 mm.

$\therefore$ Load per mm. length $= \frac{1000 \times 10^3}{600} = 1667$ N

From equation 22.8, p = K d

Also from Table 22.1, K for mild steel single roller is 8.

$\therefore$ p = 8d or 1667 = 8d ; or $d = \dfrac{1667}{8} = 208$ mm. Say 200 mm.

ILLUSTRATIVE EXAMPLE 22.2

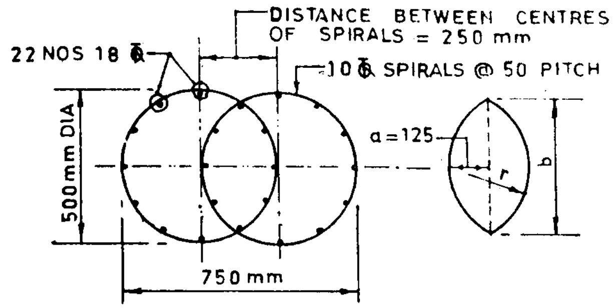
Design the spiral reinforcement for a bearing having plate size of 500 × 700 and carrying a load of 3000 KN.

Solution

Concrete stress at the base of plate = $\dfrac{3000 \times 10^3}{500 \times 700} = 8.57$ MPa

This exceeds the basic permissible compressive stress, for M20 concrete, σ_{co} = 5.0 MPa or 6.28 MPa even if a pedestal of 650 × 850 mm with dispersion grid is used. Therefore, dispersion grid with spiral reinforcement is proposed to be provided.

Two number, 500 diameter interlocked spirals as shown in Fig. 22.13, are proposed to be used.



(a) PLAN OF INTERLOCKING SPIRALS (b) AREA OF OVERLAPPING

FIG. 22·13 SPIRAL COLUMN (Example 22·2)

Area of two spirals = $2 \times \dfrac{\pi}{4} \times (500)^2 = 3{,}92{,}700$ mm^2

Deduct area of overlapping (Fig. 22.13b) = $2 \times (2/3 \times ab) = 4/3\ ab$

Again $b = 2\sqrt{a(2r - a)} = 2\sqrt{125\,(500 - 125)} = 433$

$\therefore$ Overlapping area = $4/3 ab = 4/3 \times 125 \times 433 = 72{,}170$ mm^2

$\therefore$ Nett. area of spiral = $3{,}92{,}700 - 72{,}170 = 3{,}20{,}530$ mm^2

Use 22 Nos. 16 Φ longitudinal bars in the spiral with 10 Φ spiral @ 50 mm pitch.

Load taken by the spiral column from equation 22.12,

$P = A_C \cdot \sigma_{oo} + A_s \cdot \sigma_{so} + 2\,A_{SP} \cdot \sigma_{SP}$

$A_C = 3{,}20{,}500$ mm^2 ; $\sigma_{co} = 5.0$ MPa for M20 concrete

$A_S = 22 \times 250 = 5538$ mm^2 ; $\sigma_{so} = 170$ MPa

$A_{SP} = 2 \times \pi \times 500 \times 78 = 2{,}45{,}080$ mm. unit for a length of 50 mm

$\therefore$ A_{SP} per mm. length $= \dfrac{2{,}45{,}080}{50} = 4900$ mm. units

$6_{SP} = 95$ MPa

$\therefore\ P = 3{,}20{,}530 \times 5.0 + 5588 \times 170 + 2 \times 4900 \times 95$

$= 1603 \times 10^3 + (950 + 931) \times 10^3$

$= 3484 \times 10^3$ N $= 3484$ KN > 3000 KN

Also $(A_C \cdot 6_{CO} + 2\,A_S \cdot 6_{SP})$ shall not exceed 0.5 fck. A_C (Art. 22.11.4).

$\therefore\ A_C = 3{,}20{,}530$ mm^2 $\quad \therefore\ 0.5\, fck \cdot A_C = 0.5 \times 20 \times 3{,}20{,}530 = 3205 \times 10^3$ N $= 3205$ KN

This is greater than (1603 + 931) × 10^3 i.e., 2534 KN. Hence the spiral column is sufficient to transfer the design load of 3000 KN from the bearing. The relative position of the dispersion grid and the spiral column under the bearing is shown in Fig. 22.14.

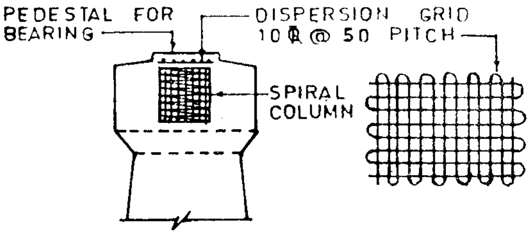


FIG. 22·14 SPIRAL AND DISPERSION GRID

22.13 PROTECTION AND MAINTENANCE OF BEARINGS

22.13.1 In a bridge structure, the bearings constitute a very important functional part on which the whole superstructure depends and therefore, they should be looked after with great care and maintained in good condition. Periodical inspection of the bearings should be made and they should be cleaned from dust, debris etc. Metallic bearings shall be greased for efficient and trouble-free service. Fig. 22.15 shows a grease-box for the protection of a metallic roller bearing.

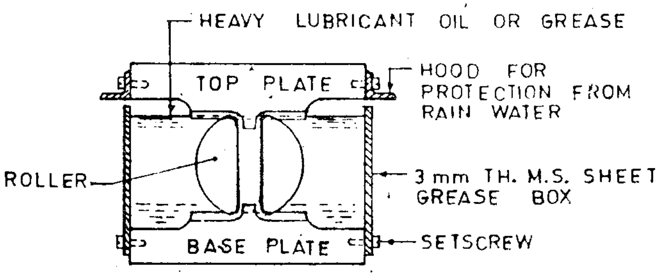


FIG. 22·15 GREASE BOX (Rokshit)[8]

22.14 EXPANSION JOINTS

22.14.1 Types of Expansion Joints

The type of expansion joint to be adopted depends on the magnitude of the expansion or contraction to be catered for. The amount of expansion or contraction again depends on the length between adjacent expansion joints. The following types of expansion joints are usually used in road bridges.

(a) Copper U-Strip & Joint Filler (Fig. 22.16a)

When the length between adjacent expansion joint is within 10 m., copper U-strip with mastic filler inside the cavity is provided. The copper U-strip allows both the expansion and contraction as well as prevents the mastic filler from falling down. This sort of expansion joint is found quite satisfactory since the movement at the ends is very small.

(b) Copper U-Strip & Joint Filler ; Edge Protection by MS Angles (Fig. 22.16b)

This type of expansion joint is adopted when the expansion length exceeds 10 m. but remains within 20 m. The angles protect the concrete edge from damage. M. S. flats attached to the angles by welding and countersunk bolts and secured to the deck concrete hold the angles in position. The fixing of the angles by m.s. bars welded to them and placed in wearing course does not stand against the impact of the moving vehicles and therefore, becomes useless after few years of service.

(c) M. S. Angles & Cover plate at Deck Level (Fig. 22.16c)

When the amount of expansion is moderately large, i.e., when the expansion length exceeds 20 m, this type of expansion joint may be used. M. S. cover plates bridge over the expansion gap having supports on the angles. The cover plate is welded to one of the angles at A in addition to fixing it with countersunk bolts and slides over the other over which it rests freely. Just beyond the free end of the plate, some gap filled up with mastic filler is kept to allow for the forward movement of the cover plate at the time of expansion. The angles are fixed to the deck concrete by m.s. flats which are again attached to the angles by counter-sunk bolts in addition to welding. It has been observed that welding alone does not function properly and the angles get detached from the anchorage bars perhaps due to improper welding or may be due to the fatigue effect caused by the constant impact of the moving vehicles. The wearing course resting on the m.s. flat will not have adequate bond for which seperation occurs and as such may get damaged. The use of epoxy resin at the interface may improve the bond. Skin reinforcement in the wearing course over the cover plate may also be provided to avoid cracking.

(d) M. S. Cover Plate over Deck Slab & Mastic Asphalt Cover (Fig. 22.16d)

This is a simpler form of type C. In this type, the angle supporting the cover plate are omitted and the cover plate is anchored into the deck concrete directly by m.s. flats attached to the cover plate by welding and counter-sunk bolts. The cover plate slides over another m.s. plate (B) which is also anchored into deck concrete by m.s. flats. Further, concrete wearing course over the cover plate is omited and replaced by mastic asphalt wearing course. The ends of the concrete wearing course are protected by m.s. skin reinforcement in addition to champhering the ends as shown in the figure.

(e) Saw-tooth Expansion Joint (Fig. 22.16e)

For spans over 50 m. to 75 m. where expansion or contraction of the deck is moderately large, saw-tooth type expansion joints are used. In this type, expansion/contraction gaps are staggered by making saw tooth type arrangement in the cover plates where the teeth are inter-woven thereby allowing traffic to cross the expansion gaps alternately laid between adjacent cover plates. The length of the expansion gap at any section is also halved due to saw-tooth arrangement. The saw-toothed plates are fixed to m.s. angles at one end and slide over the opposite angle.

(f) Finger Plate Expansion Joint (Fig. 22.16f)

This type is used when the span exceeds 75 m. The general arrangement is more or less similar to the saw tooth type expansion joint. The main difference is that in saw-tooth types, the cover plates with inter-woven saw teeth slide over the m.s. angles fixed to the deck slab. The teeth are also wider compared to the fingers of the finger plate type expansion joints. The finger plates are supported on one side, the tips remaining free as

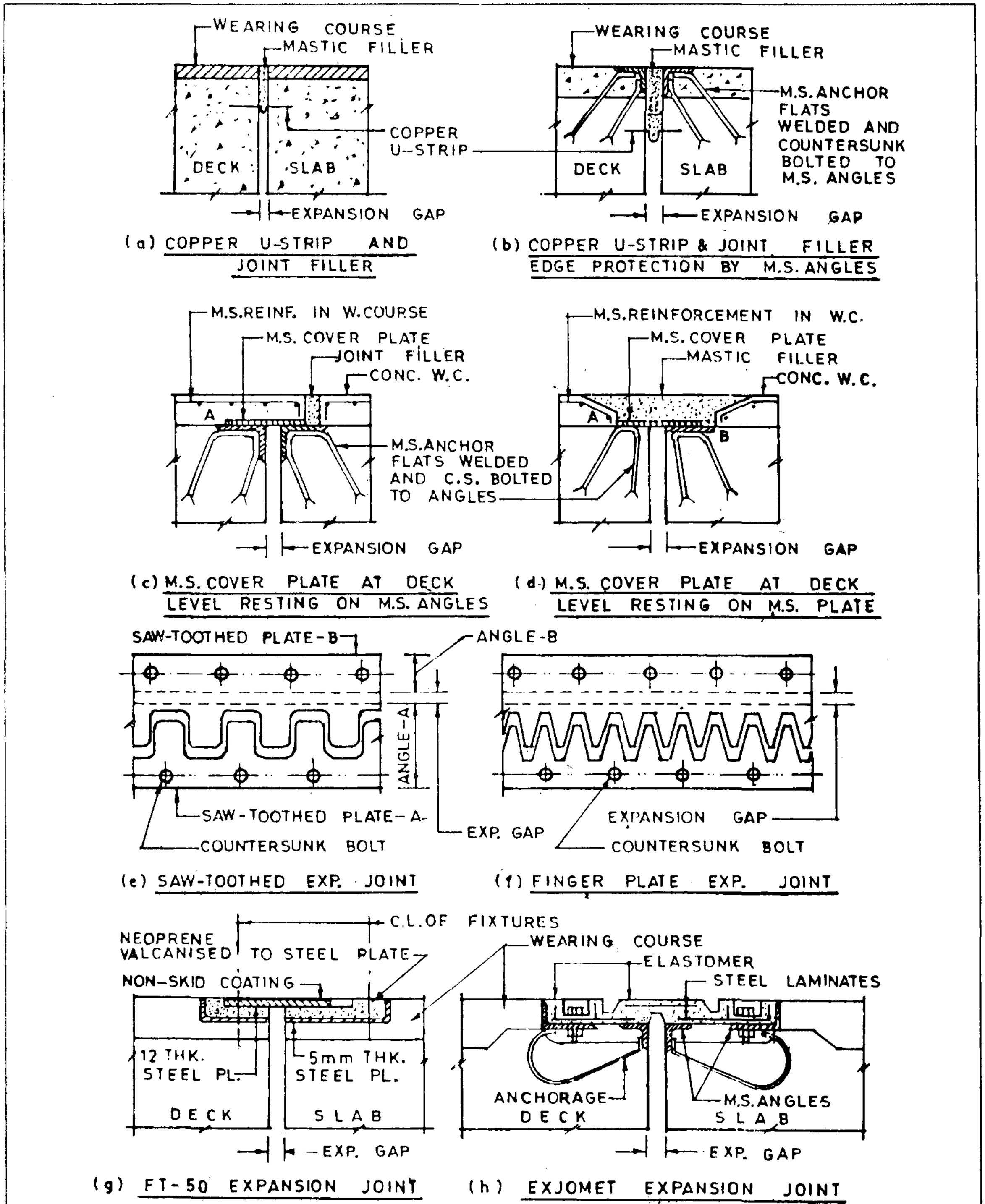


FIG. 22·16 VARIOUS TYPES OF EXPANSION JOINTS (Paracer et al)[7] (Rakshit)[9] cantilevers and as such the thickness of the finger plate is more. Expansion gaps in such joints are also more. This type of expansion joints are used in large span steel bridges.

(g) FT-50 Expansion Joint with M. S. Cover Plate over Neoprene Pads (Fig. 22.16g)

This is a ready-made expansion joint manufactured by a firm. The neoprene pad is bonded to a steel plate by valcanised process and such assembly is fixed on either end of the joint by anchor bolts. A steel cover plate is fixed at one end over the neoprene pad the other end remaining free to move as desired. A non-skid coating is applied on the top of steel plate.

(h) Exjomet Elastomeric Expansion Joint (Fig. 22.16h)

This is also a manufacturer's ready-made expansion joint. The steel insert consisting of angles is fixed to the deck concrete by anchor bars and the elastomeric cover pad in which steel plates are valcanised rides over the gap. The elastomeric pad is again fixed to the steel insert with the help of nuts and bolts.

22.14.2 Maintenance of Expansion Joints

Expansion joints, if not maintained properly, cause traffic hazards as well as restricts free flow of traffic, and therefore, proper care shall be taken to see that the defects are mended as soon as they occur. Bituminous materials used in expansion joints require to be replenished after certain time interval when either they get partially lost or they become hard due to constant service.

22.15 WEARING COURSE

22.15.1 All bridge decks are provided with some sort of wearing course, either concrete or bituminous. In new bridge decks, concrete wearing course is normally provided. When these concrete wearing course is severely damaged due to any reason what-so- ever, bituminous wearing course may be laid over the damaged concrete wearing course. (However, bituminous wearing course is provided over culverts to maintain the continuity of the black top surface along the road instead of having white surfaces of concrete wearing course at short intervals).

22.15.2 The wearing course is provided over the bridge decks to serve the following purposes :

i) To protect the structural concrete of the bridge deck from the damage caused by the moving vehicles or by rain water. The structural concrete cannot be repaired or replaced easily. This is possible in case of wearing course.

ii) A camber or cross-fall in the transverse direction of the bridge deck is necessary for easy drainage. The cross-fall for a two-lanes bridge deck can be provided easily in the wearing course laid over the finished and hardened structural concrete. The cross-fall true to camber is difficult to attain in the structural concrete. (In case of four-lanes divided carriageway, however, the cross-fall is achieved by making the structural deck down at the outer edge of the transverse deck).

22.15.3 Concrete Wearing Course

22.15.3.1 The concrete in the wearing course is made of richer grade, generally M 25 (mix. not leaner than $1 : 1\frac{1}{2} : 3$), so that it can sustain the stress and strain caused by the moving vehicle and therefore, is less susceptible to wear and tear. The concrete is to be laid in alternate panels to minimise shrinkage cracks. The sequence of concreting viz. panels 1, 2, 3, 4, 5 etc. shall be as shown in Fig. 22.17(b).

22.15.3.2 The panels shall cover the full width of the carriageway of a two-lanes bridge deck, the panel lengths being 4.0 m to 5.0 m in the longitudinal direction. Longitudinal joint at the centre of the bridge deck as shown in Fig. 22.17(a) is not recommended although, in many bridges, the longitudinal joints were provided in the past and are provided even now. The Author conducted a performance study of the concrete wearing

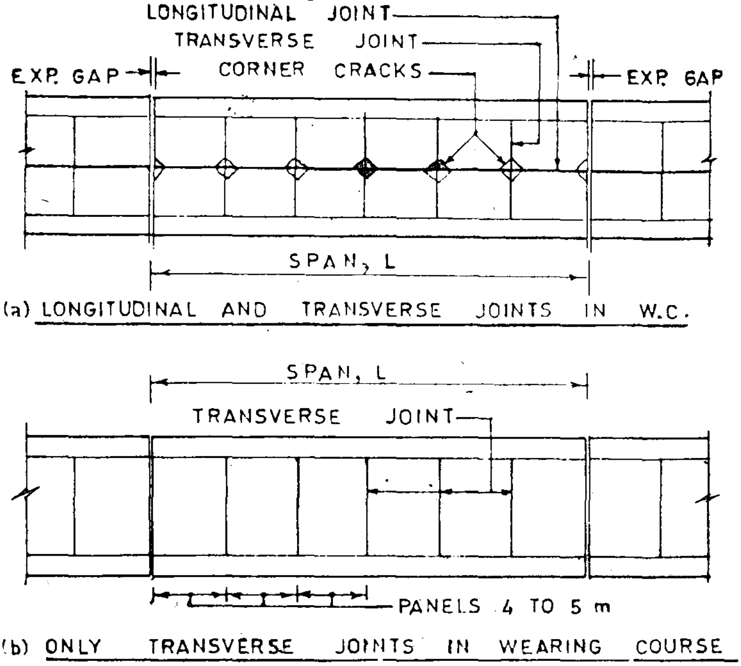


FIG. 22·17 JOINTS IN CONCRETE WEARING COURSE (Rakshit)[10]

course in 18 existing bridges in the State of West Bengal (See References at the end) and had shown that corner cracks at the junction of the longitudinal and transverse joints as shown in Fig. 22.17(a) generally develop due to lifting of the corners by warping and subsequent wheel loading at the tip.

22.15.3.3 The profile of the top of wearing course over a two- lanes horizontal deck is made parabolic with minimum thickness of 50 mm. at the outer edges and maximum thickness of 100 mm. at the centre. In case of four-lanes divided carriageway where the structural concrete is provided with a straight cross-fall (Art. 22.15.2), the wearing course may be made of 70 to 75 mm. uniform thickness.

22.15.3.4 Nominal reinforcement of 6 mm. diameter shall be provided at the top of the concrete wearing course with a cover of 20 mm. The spacing of such reinforcement shall be as specified below :

i) In simply supported slab bridges the spacing shall be 200 mm. in both directions.

ii) In simply supported slab and girder bridges, the spacing shall be generally 200 mm. in both the directions. However, cut pieces of adequate length shall be provided over the main and cross-girders in between the main bars to reduce the spacing to 100 mm. in order to take the tension to be developed at the top of the deck (Fig. 22.18a).

iii) In solid slab balanced cantilever bridges, the spacing shall be 100 mm. in the longitudinal direction in the zone near the supports and in the cantilever portion where tension may develop at the top of deck but the spacing in the transverse direction may be kept 200 mm. (Fig. 22.18b). In the central portion of the main span and in the suspended span, the spacing shall be as in (i) above.

iv) In the balanced cantilever bridges of the slab and girder type, the spacing shall be 100 mm. in both directions in the zone near the supports and in the cantilever portion where tension may develop at

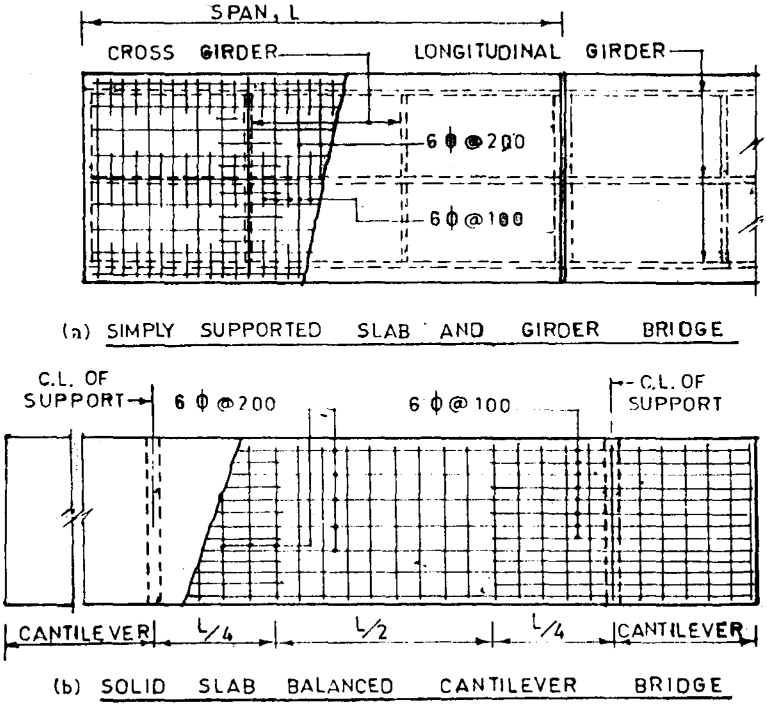


FIG. 22·18 REINFORCEMENT IN CONCRETE WEARING COURSE

the top of deck. In the central portion of the main span and in the suspended span, the spacing shall be as in (ii) above.

22.15.4 Bituminous Wearing Course

22.15.4.1 The bituminous wearing course provided in the bridge deck shall be of asphaltic concrete composed of coarse aggregate, fine aggregate, filler and binder. The specification of asphaltic concrete shall be as laid down in clause 510 of Ministry of Shipping & Transports' *"Specificaitron for Road and Bridge Works"*.

22.16 REFERENCES

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Chapter 23

RIVER TRAINING AND PROTECTIVE WORKS FOR BRIDGES

23.1 GENERAL

23.1.1 A bridge structure may fail due to the failure of its components but even if the bridge components remain safe and sound, the bridge may fail or go out of commission in the following three ways :

(a) Adverse bed scour

(b) Adverse side scour, and

(c) Scour of the approaches.

23.1.2 For bridges with deep foundations, adverse bed scour is not a problem since the foundations are designed on the consideration of maximum scour around the foundations with a minimum grip length for the stability of the foundations. Therefore, protective works against bed scour is necessary for bridges with shallow foundations where anticipated scour requires the provision of some bed protection works against the scour for the stability of the foundations.

23.1.3 Abutments and immediate approaches, may be affected by side scours against which sloped pitching with apron or sloped pitching with toe wall in case of very small scour is provided.

23.1.4 A wide flowing river in the plains has a width much in excess of what is required for normal flow. As a result, the river course often changes, sometimes flowing along left bank sometimes along right bank and sometimes in between. The waterway required for bridges over such river is much less than the river width but if a lesser length of bridge is provided having the balance width of the channel closed by approach roads, the approach may be scoured and out-flanked thereby making the bridge out of commission. In such rivers, it is necessary to provide some sort of training works to train the river in such a way that the river water is forced to flow under the bridge through the waterway provided. This procedure of forcing the rivers to flow through the designed waterway is known as ***"River Training Works"***.

23.2 PROTECTIVE WORKS OF SHALLOW FOUNDATIONS FOR MINOR BRIDGES

23.2.1 Protective Works for Open Rafts with Closed Type Abutments and Wing Walls

23.2.1.1 The open raft foundations for piers are protected from bed scour by the provision of pucca floor which may be made up of brick-on-edge over two brick flat soling set in cement mortar. Alternatively, cement

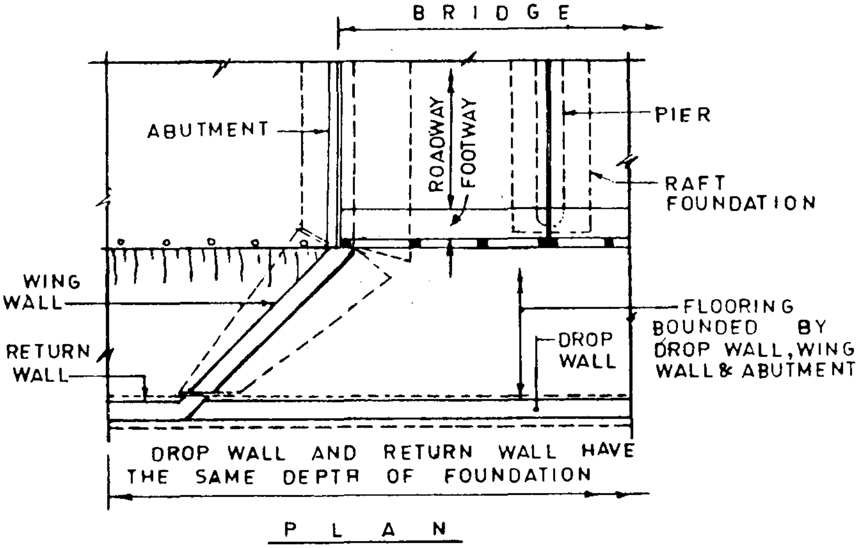


FIG. 23·1 PROTECTIVE WORKS FOR OPEN RAFT FOUNDATIONS WITH CLOSED ABUTMENTS AND WING WALLS (Rakshit)[2]

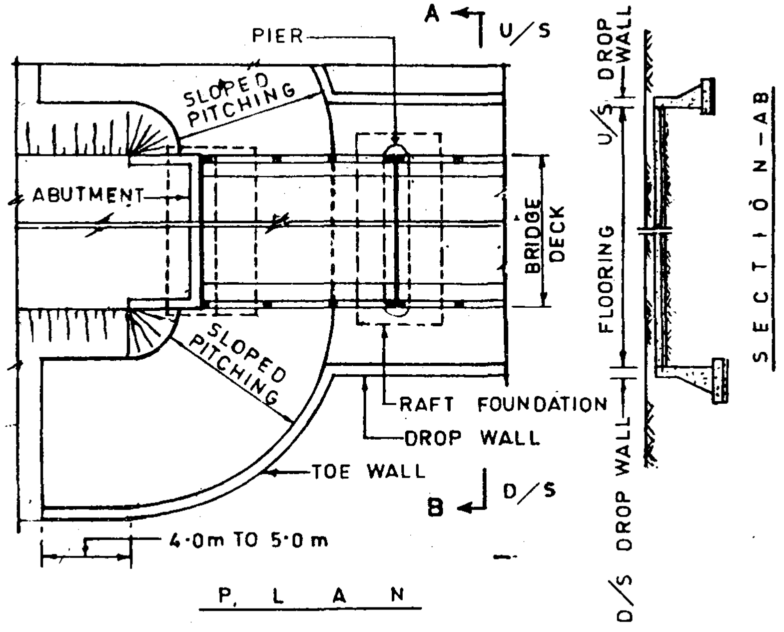


FIG. 23·2 PROTECTIVE WORKS FOR OPEN RAFT FOUNDATIONS WITH SPILL-THROUGH ABUTMENTS (Rakshit)[2]

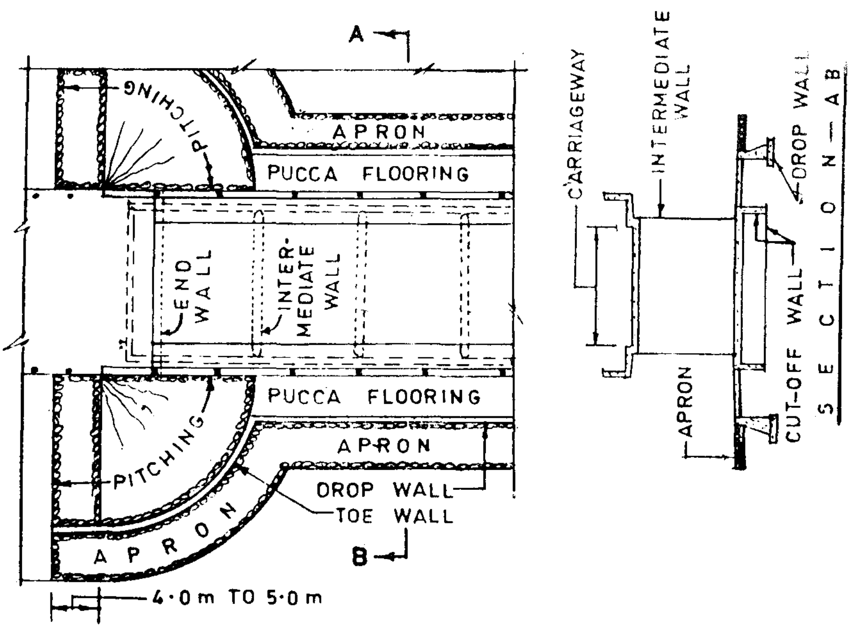


FIG.23·3 PROTECTIVE WORKS FOR MULTIPLE-BOX BRIDGES (Rakshit)[2]

concrete floor with locally available gravels or shingles may be used. The pucca flooring is again protected by drop or curtain walls provided on upstream and downstream sides (Fig. 23.1).

23.2.2 Protective Works for Open Rafts with Open Type (Spill-through) Abutments

23.2.2.1 The open raft foundations for piers are protected from bed scour as described in Art. 23.2.1.1. The front and side slopes around abutments are protected with pitching which may consists of cement-brick blocks where bricks are cheaper or may consist of cement concrete blocks or stone boulders where stone materials are locally available at cheaper rates (Fig. 23.2). The pitching is again protected by providing toe walls.

23.2.3 Protective Works for Multiple-box Bridges

23.2.3.1 The front and side slopes around the end walls of the multiple-box structures are also protected from scour with pitching similar to the arrangement made around spill-through abutments as described in Art. 23.2.2.1. The pitching is protected by toe wall. Where the bed scour is more, launching aprons are provided in front of the drop walls (Fig. 23.3). The design of this apron shall be made as described in Art. 23.3.2.7 assuming d (max.) = (1.5 dm-x).

23.3 PROTECTIVE WORKS FOR MAJOR BRIDGES

23.3.1 For Bridges with Waterway from High Bank to High Bank

23.3.1.1 The foundations for such major bridges are deep and as such no bed protection is necessary. The protection of the slope of the spill-through abutments shall, however, be made as in Fig. 23.2 or 23.3.

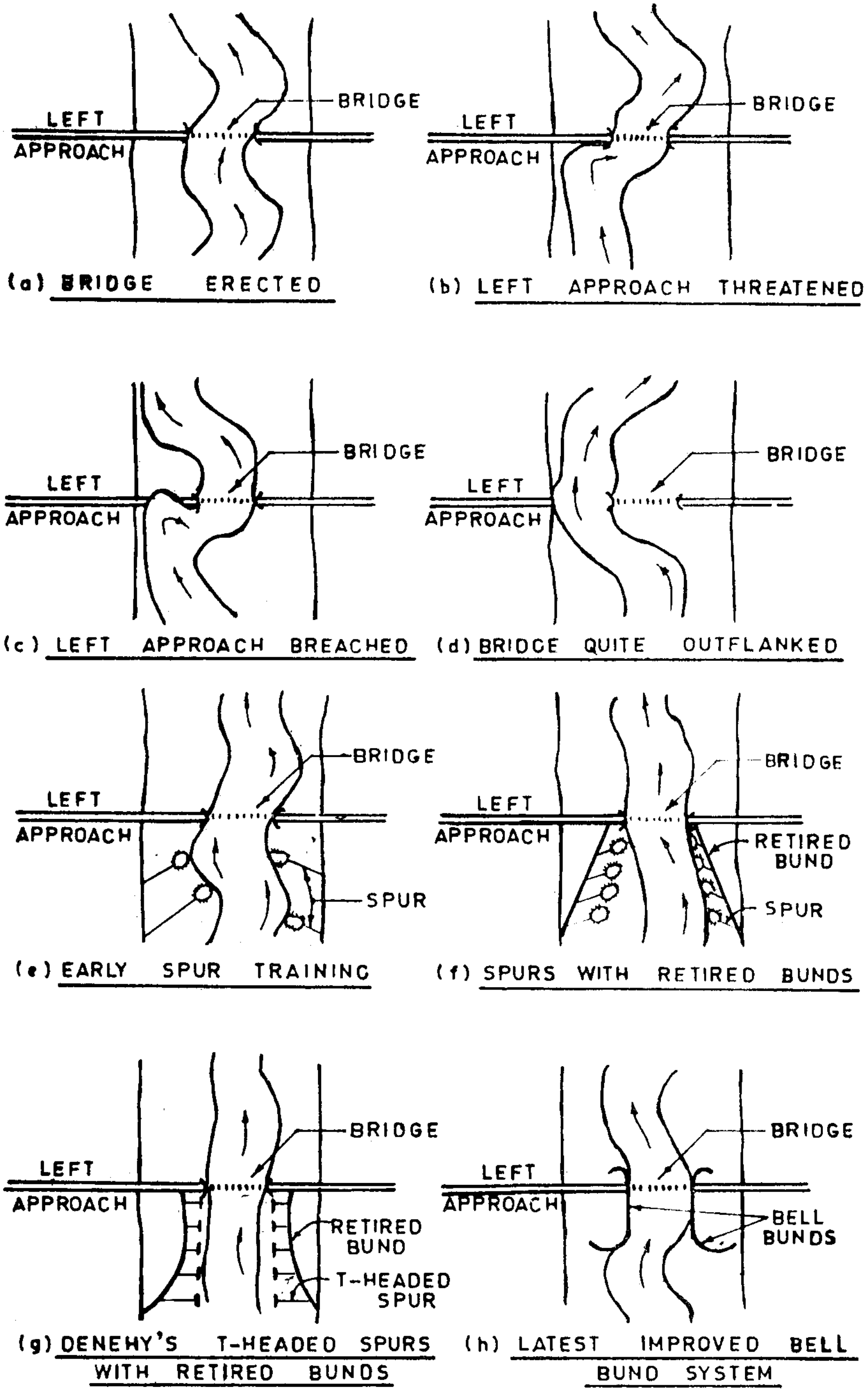


FIG.23·4 DEVELOPMENT OF MODERN RIVER TRAINING SYSTEM (Spring)[4]

23.3.2 For Bridges with Waterway much less than the Width from High Bank to High Bank

23.3.2.1 The rivers in alluvial plains are sometimes very wide. During dry season, the flow is restricted to a very small width. Even in flood season, the entire width is not covered by flowing water. If it does so and the full width is covered by flood water, the depth of flow is very shallow. The flood discharge in these rivers is such that a part of the channel is enough to carry the flood discharge. That is, if the river is constricted and a bridge of smaller length than the river width is provided, it is possible for the constricted channel to carry the flood discharge since even in the constricted channel, the area of cross-section of the channel at H. F. L. will be maintained more or less the same by scouring the bed and deepening the channel. Generally, such constriction of channel may be up to 30 to 35 percent of the full width. For example, the length of Teesta Bridge near Jalpaiguri Town (West Bengal) is 1004 m whereas the width of the channel between high banks is 3050 m., i.e., the constriction of the channel is 33 percent. The Damodar Bridge near Burdwan Town (West Bengal) has a bridge length of 506 m. in place of the river width of 1600 m. In this case, the constriction of the channel is 32 percent. Such constriction of the channel is only possible if some measures are adopted so as to guide the flow through this constricted channel. These measures are discussed in the following paragraph.

23.3.2.2 ***Development of the Guide Bund (or Guide Bank) System***

The development of the modern river training works by the provision of guide bunds is shown in Fig. 23.4. In a wide river, if a bridge is constructed by restricting the channel width without any training works (Fig. 23.4a) the river flow will have a tendency to meander and ultimately attack the approach embankments built within the high banks as shown in Fig. 23.4b and 23.4c. There is every possibility of the bridge being outflanked and remaining out of commission as shown in Fig. 23.4d. To prevent the meandering tendency of the channel flow, early method of river training was by provision of spurs (Fig. 23.4e). An improved method was used later on by providing spurs with retired bunds (Fig. 23.4f). In both these methods, heavy pitching was required to protect the shank and head of the spurs. A still improved version of using spurs for river training is the Denehy's T-headed spurs (Fig. 23.4g). These spurs are ordinary spurs with retired bunds having an arm on the river side parallel to the flow. These spurs required less quantity of stone for pitching. The modern system of river training by provision of guide banks or guide bunds was evolved by J. R. Bell and hence, these guide bunds are something called Bell bunds. Guide bunds are two embankments more or less parallel to the high banks of the river. These embankments with their ends curved are properly protected or armoured with stones. The curved head of the bunds are provided to guide the flow through the bridge and hence, these bunds are termed as guide bunds (Fig. 23.4h).

23.3.2.3 ***Design Principles of Guide Bunds***

Fig. 23.5 shows how guide bunds guide the flow thorough the bridge. The flow has a tendency to attack the approach road as in bridges without training works (Fig. 23.4b) but the situation like Fig. 23.4c cannot be created as the flow must pass through the bridge in a round about way having entry along the curved head. It is the length of the guide bund which keeps the flow away from the approach embankment thereby saving the possible attack and ultimate outflanking of the approaches. The guide bunds maintain a safe distance between the approach embankments and the possible embayments. The curved heads guide the water flowing through the Khadir (i.e., the width over which the river meanders during high floods) into the constricted channel. The curved tails ensure that the river does not attack approach embankments.

23.3.2.4 ***Length of Guide Bunds***

The length of guide bunds on the upstream side is normally kept as 1.0L to 1.5L (Fig. 23.6) for straight guide bunds which are generally preferred as it is found that parallel straight guide bunds give uniform flow from the head of the guide bund to the axis of the bridge. The length of guide bunds on the downstream side is normally 0.2 L where L is the length of the bridge as shown in Fig. 23.6.

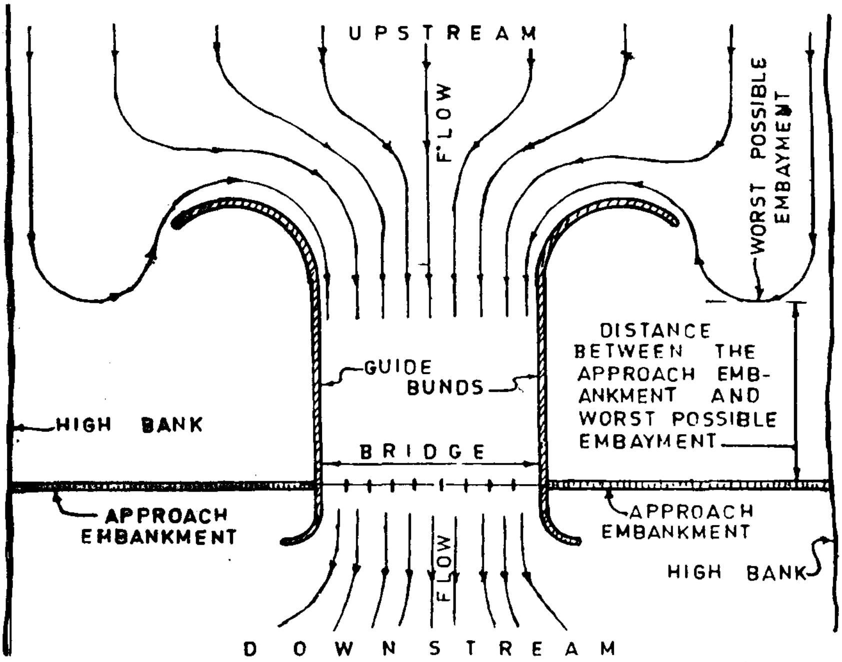


FIG. 23·5 HOW GUIDE BUNDS GUIDE THE FLOW THROUGH THE BRIDGE (IRC)[1]

23.3.2.5 *Radius for Curved Head & Tail of Guide Bunds (Fig. 23.6)*

Radius of curved head is generally between 0.4 to 0.5 times the length of the bridge between abutments but it shall not be less than 150 m nor more than 600 m unless required from model studies. Radius of curved tail is from 0.3 to 0.4 times the radius of the curved head.

23.3.2.6 *Sweep Angles (Fig. 23.6)*

The sweep angle for the curved head is 120 to 140 degrees while the same for the curved tail is 30 to 60 degrees.

23.3.2.7 *Design of Guide Bunds*

(a) *Top Width :* The top width of guide bunds is generally provided such that materials can be brought to site by trucks. A width of 6.0 m is found to be adequate for this purpose.

(b) *Free Board :* The minimum free board from the pond level (i.e., the level of water behind the guide bunds) to the top of the guide bunds shall be 1.5 m to 1.8 m. The water in the pond remains still the level of which is the level of water at the head of the guide bunds including afflux. The same free board shall also be maintained for the approach embankment also since the pond level is the same.

(c) *Side Slopes :* The side slopes of the guide bunds shall be determined from the consideration of stability of slopes of the embankments as well as from the consideration of hydraulic gradients. Generally, a side slope of 2(H) to 1(V) is adopted for predominantly cohesionless soils. Side slopes of 2.5 (H) to 1 (V) or 3.0 (H) to 1 (V) are also used as required from the considerations stated above.

(d) *Slope Protection :* The river side slope of guide bunds shall be protected with pitching against the onrush of flow. The pitching shall be extended up to the top of guide bunds and taken at least 0.6 m. inside the top

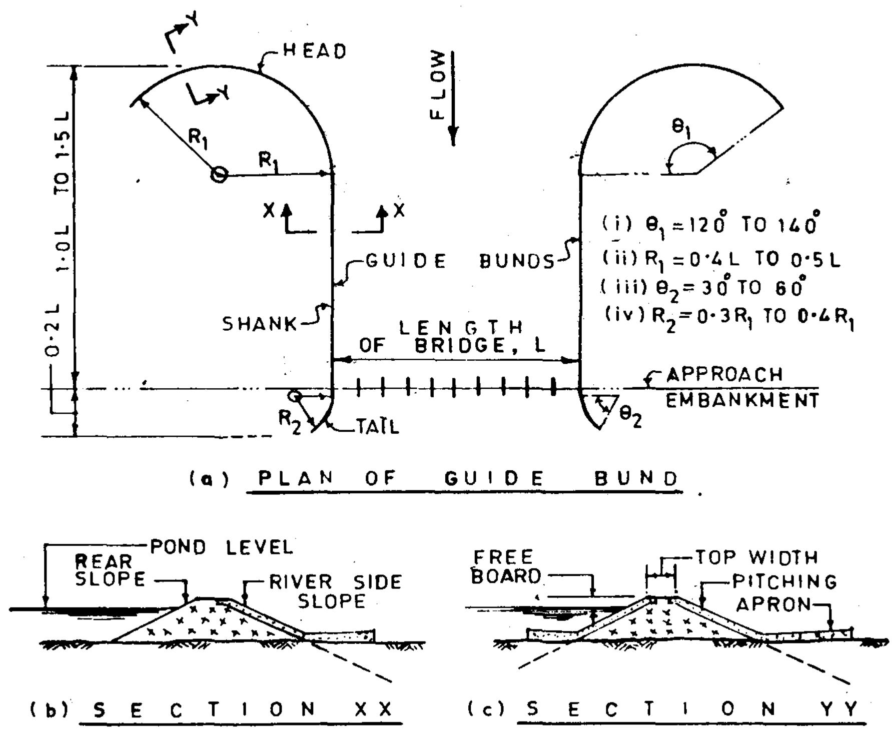


FIG.23·6 DIMENSIONS AND COMPONENTS OF GUIDE BUND (IRC)[1]

width. Rear side slopes of guide bunds are not subjected by the direct attack of the river flow. These are only subjected to the wave splashing of the pond water and as such 0.3 m to 0.6 m thick cover of clayey or silty earth with turfing will be adequate unless heavy wave action is anticipated in which case light stone pitching up to 1.0 m above pond level shall be done. The pitching on the river side may be done by cement concrete blocks or individual stones or stones in wire mesh crates.

(e) *Size and Weight of Stones for Pitching :* The size of stones in individual stone pitching to withstand the on rush of flow is given by :

$$d = KV^2 \qquad (23.1)$$

Where,

d = Equivalent diameter of stones in metres.
K = A constant having values of 0.0282 for side slope of 2 : 1 and 0.0216 for side slope of 3 : 1.
V = Mean design velocity in m/sec.

Table 23.1 gives the size and weight of stones for velocities up to 5.0 m/sec, assuming the specific gravity of stone as 2.65.

TABLE 23.1 : SIZE AND WEIGHT OF STONES FOR PITCHING FOR VARIOUS VELOCITIES & SIDE SLOPES *(IRC)*[1]

Mean Design Velocity m/sec	Minimum Size & Weight of Stone			
	Slope 2 : 1		Slope 3 : 1	
	Diameter (cm)	Weight (Kg)	Diameter (cm)	Weight (Kg)
Up to 2.5	30	40	30	40
3.0	30	40	30	40
3.5	35	59	30	40
4.0	45	126	35	59
4.5	57	257	44	118
5.0	71	497	54	218

NOTES :

(1) No stone weighing less than 40 Kg shall be used.

(2) Where the required size of stones are not economically available, cement concrete blocks or stones in wire crates may be used as isolated stones of equivalent weight. Cement concrete blocks are preferable.

(f) *Thickness of Pitching :* The thickness, T, of pitching may be worked out from equation 23.2 as given below subject to a minimum value of 0.3 metre and maximum value of 1.0 metre.

$$T = 0.06\,(Q)^{1/3} \qquad (23.2)$$

Where,

T = Thickness in m
Q = Design discharge in m^3/sec

The thickness of pitching, however, shall be increased suitably for guide bunds to be provided for bridges across major rivers.

(g) *Filter Design :* Suitably designed filter is necessary under the slope pitching in order to prevent the loss of embankment materials through the pores of stone pitching/cement block pitching/stone crate pitching. The filter will also allow escape of seepage water without creating any uplift pressure on the pitching. For details, reference may be made to ***"IRC : 89-1985 – Guidelines for Design and Construction of River Training and Control Works for Road Bridges"***.

(h) *Size and Weight of Stones for Launching Aprons*

The size and weight of stones for launching aprons may be determined from equation 23.3 as given below :

$$d = 0.0418\,V^2 \qquad (23.3)$$

Where,

d = Equivalent dia, of stone in m
V = Mean design velocity in m/sec.

Table 23.2 given the size and weight of stones to be used in launching aprons for velocities up to 5.0 m/sec. assuming the specific gravity of stone as 2.65.

TABLE 23.2 : SIZE & WEIGHT OF STONES FOR LAUNCHING APRONS FOR VARIOUS VELOCITIES *(IRC)*[1]

Mean Design Velocity (m/sec.)	Minimum Size & Weight of Stone	
	Diameter (cm)	Weight (Kg)
Up to 2.5	30	40
3.0	38	76
3.5	51	184
4.0	67	417
4.5	85	852
5.0	104	1561

NOTES :

(1) No stone weighing less than 40 Kg shall be used.

(2) Where the required size of stones are not economically available, cement concrete blocks or stones in wire crates may be used as isolated stones of equivalent weight, preference being given to cement concrete blocks.

(i) *Shape and Size of Launching Apron :* The width of launching apron is generally made equal to 1.5 d (max) (Fig. 23.7) where d (max) is the maximum anticipated scour level from L. W. L. The value of d (max) shall be determined from Table 23.3.

TABLE 23.3 : VALUES OF d(max) *(IRC)*[1]

Sl. No.	Location	Deepest Known Scour Depth from H. F. L.	d(max) from L. W. L.
1.	Upstream curved head of guide bund	2.0 to 2.5 dm.	(2.0 to 2.5 dm-x)
2.	Straight portion of guide bund including downstream curved tail of guide bund	1.5 dm	(1.5 dm-x)

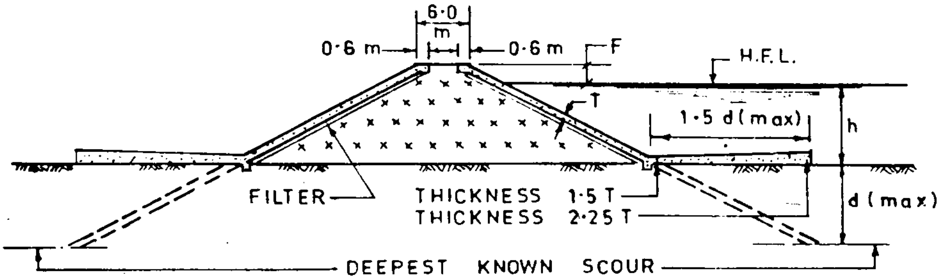
NOTES :

(1) The value of dm is determined from equation 3.17.

(2) x = level difference between H. F. L. and L. W. L. in metres.

The thickness of launching apron at inner end may be kept as 1.5 T and at outer end as 2.25 T as shown in Fig. 23.7. The slope of launching apron is generally taken as 2:1 for loose stones and 1.5 : 1 for cement concrete blocks or stones in wire crates.

(I) *Wire Crates in Slopes or in Apron :* The wire crates shall be made from 5 mm galvanised iron wire. The mesh size shall be 150 mm. The size of wire crates for shallow and accessible locations shall be 3.0 m × 1.5 m ×



Notes

(i) F = HEIGHT OF TOP OF GUIDE BUND FROM H.F.L.(m)

(ii) h = LEVEL DIFFERENCE BETWEEN H.F.L. AND L.W.L.(m)

(iii) T = THICKNESS OF PITCHING (m)

(iv) VOLUME OF STONE IN SLOPE PITCHING PER METRE LENGTH = 2·25 (F + h) T

(v) VOLUME OF STONE IN APRON PER METRE LENGTH = 2·81 d(max). T

FIG. 23·7 DETAILS OF LAUNCHING APRON

1.25 m. The crates shall be divided into 1.5 m long compartments by cross-netting if there is a chance of overturning of the crates after they are laid. Maximum and minimum sizes of wire crates shall be 7.5 m × 3.0 m × 0.6 m and 2.0 m × 1.0 m × 0.3 m respectively. When crates are large, the sides shall be securely tied at intervals to prevent bulging.

ILLUSTRATIVE EXAMPLE 23.1

A bridge is to be constructed over a river in alluvial plains having a width between high banks, i.e., Khadir width of 1600 m. and a design discharge 16,000 m³/sec. State whether guide bunds are necessary to train the flow of the river and if so, design the guide bunds. Design velocity = 4.0 m/sec. H. F. L. = 33.30 m, L. W. L. = 25.10 m. Silt factor of bed materials, f = 1.25.

Solution

From equation 3.18, linear waterway required for the bridge $= C\sqrt{Q} = 4.8\sqrt{16{,}000} = 607$ m.

Adopt 11 spans of 46.0 m. $\therefore \quad W = 11 \times 46.0 = 506$ m. $= L$

The width of Khadir = 1600 m. Hence guide bunds are necessary to guide the flow through the bridge.

Length of guide bund :

From Art. 23.3.2.4, length of guide bund upstream of bridge from bridge axis is 1.0 to 1.5 L. Let us take a value of 1.30 L, i.e., 1.30 × 506 = 658 m. Length of guide bund on the downstream side = 0.2 L = 0.2 × 506 = 102 m.

Total length of guide bound = 658 + 102 = 760 m.

Radius of Curvature of Head & Tail

From Art. 23.3.2.5,

Radius for upstream head = 0.4 L to 0.5 L. Let us adopt a value of $R_1 = 0.45$ L = 0.45 × 506 = 228 m.

Radius of tall, $R_2 = 0.4\ R_1 = 0.4 \times 228 = 91$ m.

Sweep Angles

Adopt sweep angle of upstream head as 130° and of downstream tail as 45°.

Top Width, Fee-board, Side Slopes etc.

Make	Top Width	=	6.0 m
	Free-board	=	1.6 m
	Side Slopes	=	2.0 (H) : 1.0 (V)

Size and Weight of Stones for Pitching

From Table 23.1 for design velocity of 4.0 m/sec & side slope of 2 : 1, dia. of stone = 45 cm and weight = 126 Kg. Stones of such size is difficult to procure economically and also to handle. Therefore, cement concrete block may be cast at site.

Make the size of block = 0.5 m × 0.5 m × 0.3 m. Weight = 0.5 × 0.5 × 0.3 × 2200 = 165 Kg > 126 Kg.

Thickness of Pitching

From equation 23.2, $T = 0.06\,(Q)^{1/3} = 0.06\,(16{,}000)^{1/3} = 1.51 \text{ m}$

But as per Art. 23.3.2.7 (f), maximum thickness of pitching shall be 1.0 m. Hence adopt this value.

Size and Weight of Stones for Launching Apron

From Table 23.2, size of stone for design velocity of 4.0 m/sec = 67 cm and weight = 417 Kg. The size being too large is not economically available. Therefore, cement concrete blocks are proposed to be used. The thickness of block will vary from 1.5 T to 2.25 T (Fig. 23.7) and Art. 23.3.2.7 (h).

i.e., thickness will vary from 1.5 × 1.0 to 2.25 × 1.0, i.e., 1.5 m to 2.25 m.

Make the block 0.75 m × 0.75 m in plan.

Therefore, minimum weight of each block = 0.75 × 0.75 × 1.5 × 2200 = 1856 Kg > 417 Kg.

Maximum weight of the block at outer end = 0.75 × 0.75 × 2.25 × 2200 = 2785 Kg.

Hence satisfactory.

Shape and Size of Launching Apron

From Art. 23.3.2.7(i), width of launching apron = 1.5 d(max) ; x = H. F. L. – L. W. L. = 33.30 – 25.10 = 8.2 m.

From Table 23.3, d(max) from L. W. L. —

(i) at upstream curved head = [2.25 (av.) d_m – x]

(ii) at straight portion guide bund and at downstream curved tail = (1.5 dm-x)

dm from equation 3.17 is $dm = 1.34\left[\dfrac{q^2}{f}\right]^{1/3}$

Effective waterway from equation 3.19 = L – nt = 506 – 10 × 1.0 = 496 m.

$$q = \frac{Q}{L} = \frac{16{,}000}{496} = 32.26 \text{ m}^3/\text{sec}. \quad f = 1.25 \text{ (given)}$$

$$\therefore\ dm = 1.34\left[\frac{(32.26)^2}{1.25}\right]^{1/3} = 12.61 \text{ m}$$

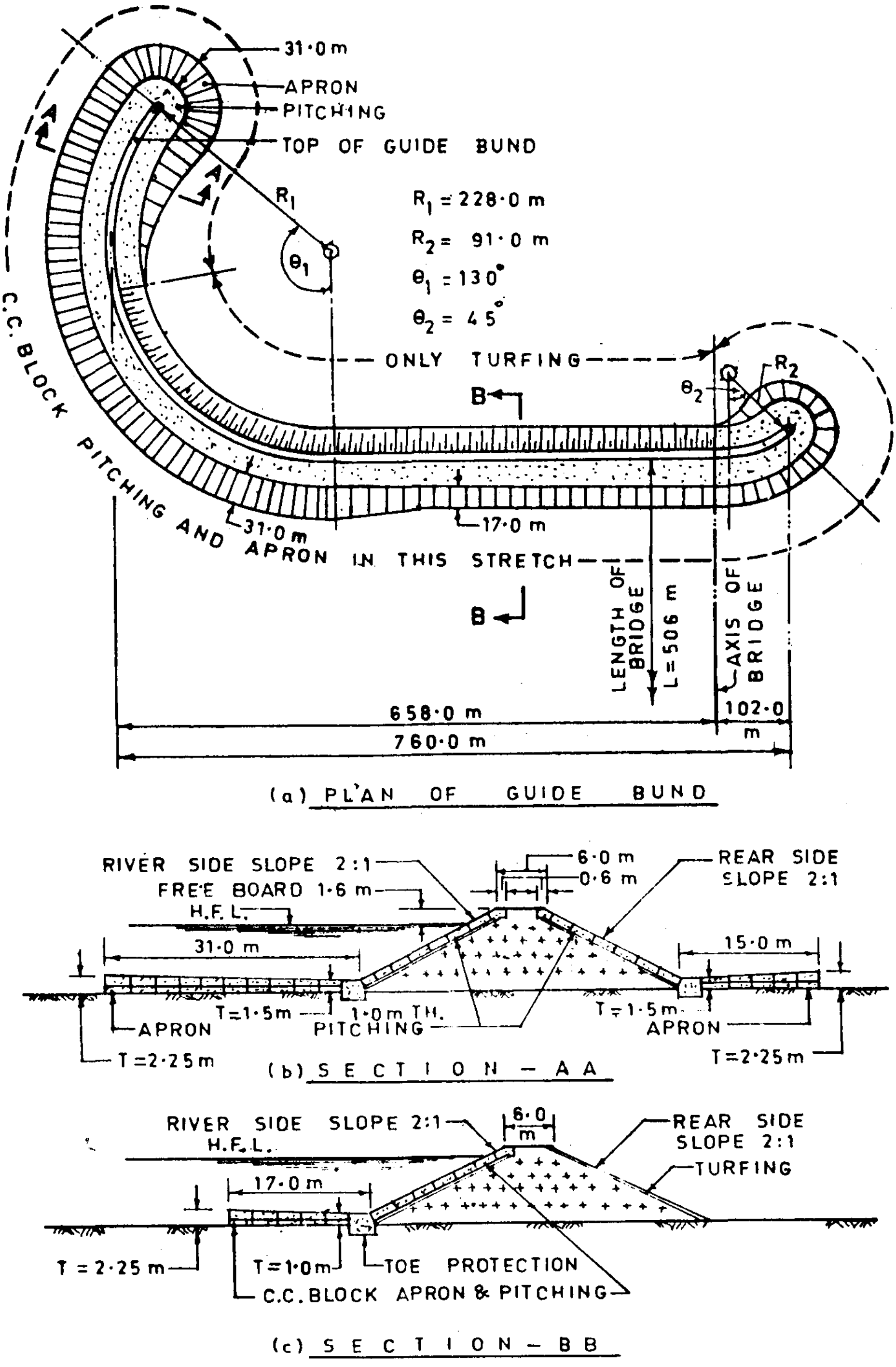


FIG. 23·8 DETAILS OF GUIDE BUND (Ex. 23·1)

$\therefore$ d(max) at upstream of curved head

= (2.25 dm-x) = 2.25 × 12.61 − 8.2 = 20.17 m

$\therefore$ Width of apron at upstream head = 1.5 d (max) = 1.5 × 20.17 = 30.26 Say 31.0 m

d(max) at straight portion and at downstream tail = $(1.5dm - x)$

$= (1.5 \times 12.61 - 8.2) = 10.72$ m

$\therefore$ Width of apron at straight portion and at downstream tail = $1.5\ d(max)$

$= 1.5 \times 10.72 = 16.08$ Say $17.0\ m$.

Thickness of apron at junction with slope = $1.5\ T = 1.5 \times 1.0 = 1.5$ m

Thickness of apron at outer end = $2.25\ T = 2.25 \times 1.0 = 2.25$ m

The details of the guide bund are shown in Fig. 23.8.

23.4 REFERENCES

1. IRC:89-1985 : Guidelines for Design and Construction of River Training and Control Works for Road Bridges — Indian Roads Congress.
2. Rakshit, K. S. — *"The Protection of Culverts and Minor Bridges against Damages caused by the Effect of Scour or Seepage Flow"* — Indian Highway (April, 1984), — Indian Roads Congress.
3. Ramakrishna Rao, C. — *"River Training for Bridges by the Guide Bank System"* — Transport Communication Monthly Review (July, 1961) — Indian Roads Congress.
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CHAPTER 24

CONSTRUCTION, ERECTION AND MAINTENANCE OF BRIDGES

24.1 GENERAL

24.1.1 The *"construction"* of a bridge involves the building of its various components such as foundation, substructure, superstructure and other ancillary works which include the construction of the immediate approaches, finishing works and protective works for the bridge. The *"erection"* of a bridge means the installation of the precast or prefabricated superstructure units at their designed locations. The units may be cast or fabricated in a factory or at the approaches to the bridge or at ground, bed level or in a casting yard nearby and brought to the design locations by various means such as carriage by trucks, barges, lifting by derricks or cranes or launching by launching truss and the like. The *"maintenance"* of a bridge is the general up-keep of the bridge by routine maintenance or special maintenance to be done on the basis of routine inspection of the bridge. The construction aspects have been partially discussed in the preceding chapters dealing with the various components by bridge structures or various types of bridges. Therefore, in this chapter main stress is given on the erection and maintenance aspect of the bridge.

24.2 CONSTRUCTION OF BRIDGES

24.2.1 Components of Bridges to be Constructed In Stages

24.2.1.1 A bridge has the following components which require to be constructed in stages, one after the other. The construction of the substructure is dependent on the completion of the foundation. Similarly, the construction of the superstructure is dependent on the completion of the substructure. Therefore, the construction of these components shall be taken up in proper sequence.

24.2.1.2 ***Components of Bridges***

(a) Foundation
(b) Substructure-piers, abutments and wing walls
(c) Bearings
(d) Superstructure —
(i) Girders
(ii) Deck Slab
(iii) Wheel guard, footway slab, railings and wearing course.

24.2.2 Ancillary Works for Bridges to be Taken Up Simultaneously

24.2.2.1 The ancillary works for bridges such as the earthwork and protective works for the approaches, and guide bunds etc. shall be taken up along with the m.' ı bridge works.

24.2.3 Main Items of Works for Bridges as Necessary (not shown in proper sequence)

24.2.3.1 *For Reinformed and Prestressed Concrete Bridges*

(i) Providing piles and pile cap.

(ii) Providing base of foundation by making island, cofferdam etc. in shallow depth of water and laying cutting edge for well foundation.

(iii) Floating and grounding caissons in deep water.

(iv) Sinking, plugging and sand filling in wells including well cap.

(v) Form-work for foundation and form-work and centering for substructure.

(vi) Bending and placing reinforcement in foundation and substructure.

(vii) Masonry and concrete works in foundation and substructure.

(viii) Centering, form-work, placing reinforcement and casting girders (for PSC girders after placing of H.T. cables) or casting arch components.

(ix) Casting deck slab over girders (for PSC girders after erection/launching if necessary).

(x) Fixing bearings/hinges for arches/expansion joints etc.

(xi) Miscellaneous works such as W.C., railings, earth work in approaches, protective works, guide bunds (if any) etc.

24.2.3.2 *For Steel Bridges*

(i) Providing base for foundation by making island, cofferdam etc. in shallow depth of water, laying cutting edge for well foundation.

(ii) Floating and grounding caissons in deep water.

(iii) Form-work foundation and form-work & centering for substructure & superstructure (if any).

(iv) Bending and Placing reinforcement in foundation, substructure and superstructure (if any).

(v) Masonry works in foundation & substructure.

(vi) Concrete works in foundation, substructure and superstructure (if any).

(vii) Steel works in superstructure

(viii) Fabrication and erection of girders, trusses, arches (for cable-stayed or suspension bridges after fixing cables, suspenders etc.).

(ix) Providing concrete or steel decking as necessary.

(x) Fixing bearings/hinges for arches/expansion joints.

(xi) Miscellaneous works such as W.C., railings, earthwork in approaches, protective works, guide bunds (if any) etc.

24.2.4 Any bridge construction involves the undernoted broad subheads to execute the works :

(a) Site office, godown etc. and personnel to manage these establishments.

(b) Construction materials and working drawings required to build the components of the bridge.

(c) Machinery, T & P, helper materials etc. to implement the works.

(d) Labours — both skilled and unskilled to execute the works.

(e) Technical personnel to supervise the works.

Therefore, a good planning shall be made at the outset for the construction of site-office, godown etc., collection of materials, working drawings, machinery, T & P, helper materials, mobilisation of labour, in a phased manner as and when their services are required etc. For this purpose, a PERT/CPM planning technique helps a lot as these methods indicate the inter- dependability of the various activities and also the critical path or the critical activity which is the deciding factor for completion of the project in time.

After the construction of site office, godown etc. and the collection of materials and working drawings for the foundations, this work may be taken up first. Other works may be undertaken in proper sequence as per programme made in this regard. All items of works shall be carried out as per drawings, specifications and sound engineering practices.

24.2.5 Before describing the bridge construction works item wise, the general items of works which are involved in the construction of foundation to superstructure, namely (a) form- work and centering, (b) bending and placing reinforcement, (c) concreting, (d) curing of concreting etc. are discussed.

24.2.6 Form-work and Centering

The forms shall be strong enough to bear the loads of concrete and workmen, the liquid pressure of the freshly laid concrete and the impact effect of ramming or vibration. The form-work shall be sufficiently water-proof so as to prevent the absorption of water from concrete or the leakage of cement slurry the effect of which is the porosity and honey-combing of concrete. The form-work shall be true to line and levels. The forms shall be easily removable from the concrete surface without damage. For this purpose, the forms may be coated with a thin layer of mineral oil, soft soap proprietory solution or white wash.

The centering props shall be able to take the dead load of the concrete including construction live load. The foundation on which the props rest shall also be adequately made safe to take the loads coming over it. The centering shall be braced both longitudinally and transversely along with diagonal bracings.

The removal of forms shall be done when concrete has attained sufficient strength. The following are the general time-schedule for removal of form-work and props for cement concrete where ordinary portland cement is used.

(a)	Form-work for vertical faces of all structural members	...	1-2 days
(b)	Form-work for slab (with props left under)	...	3 days
(c)	Form-work for beams (with props left under)	...	7 days
(d)	Props under slab		
	(i) For spans up to 4.5 m	...	7 days
	(ii) For spans over 4.5 m	...	14 days
(e)	Props under beams and arches —		
	(i) For spans up to 6.0 m	...	14 days
	(ii) For spans over 6.0 m	...	21 days

24.2.7 Bending and Placing Reinforcement

All reinforcement shall be made free from scales, rust, coats of paint, oil, mud etc. before bending. Bar bending requires skilled workmanship. Bending of bars may be done by fixing the bar between two iron pins driven in wooden platform. The required radius of curvature of a bar can be obtained by bending the bar round the mandrel of required diameter fixed in the iron pin. The force required for bending bars is applied by a lever made out of hollow pipe. Bars shall be bent in cool condition.

The placing and fixing of reinforcement shall be as per approved detail drawings with adequate cover which may be maintained by using precast blocks of appropriate thickness made of cement-sand mortar of 1:2 proportion with a twisted binding wire at centre. During placing of reinforcement, these supports are tied by the wire with the reinforcement.

Laps shall be adequate in length and staggered as far as possible to minimise weakness at any section.

24.2.8 Concreting

24.2.8.1 Concrete shall be made with graded coarse aggregates, sand of appropriate fineness modulus, good water and fresh cement. The water-cement ratio shall be as low as possible from the strength consideration but from practical consideration, namely, workability, a reasonable value of W/C ratio shall be adopted. The concrete mix must be workable as otherwise the concrete will be porous and honey-combed. Porous concrete absorbs more moisture from atmosphere and this moisture becomes an electrolyte and a source of corrosion.

24.2.8.2 The concrete must be mixed in a mixer machine after weigh-batching all the ingredients of the concrete. The quantity of moisture present in the coarse and fine aggregates, shall be determined frequently and the quantity of water to be added in the mix shall be adjusted accordingly taking into consideration the quantum of water present in the aggregates so as to maintain the design W/C ratio uneffected.

24.2.8.3 In bridge construction, controlled concrete is used. ***"Controlled concrete"*** means the concrete which is controlled at every stage and in which the proportions of cement, fine aggregate, coarse aggregate, water and admixture (if any to increase the workability) are predetermined in the laboratory by weight on the basis of the target strength and required workability. ***"Design of Mix"*** is the most important item of controlled concrete. Design of mix means the determination of the quantities of the ingredients in the mix with a view to achieving the target-mean strength for each grade of concrete. *"IRC:21-1987—Section III—Cement Concrete (Plain and Reinforced) and IRC: 18-1985 (Post-tensioned concrete)"* specify the target- mean strengths against each standard grades of concrete.

24.2.8.4 For sampling and testing of concrete, statistical approach shall be adopted. A random sampling procedure shall be followed so that each concrete batch shall have a reasonable chance of being tested. The minimum frequency of sampling of each grade of concrete shall be one 150 mm test cube for each two cubic metres of concrete for the first 300 m^3 of concrete or concrete in the first major span of bridge whichever is less to be reduced to one test cube for every 3 m^3 for subsequent works.

24.2.8.5 Concrete shall be placed in the actual work soon after its mixing so that no initial setting does take place. The placing of concrete shall be done carefully so that no seggregation of the ingredients neither the displacement of the reinforcement does occur for concreting lower part of a deep structure such as bottom flange and lower part of the web of PSC girders having depth over 2.0 metres Drop chutes attached with hoppers shall be used and concrete lowered in such case.

24.2.8.6 To produce a uniform, dense concrete, proper and controlled compaction is also very essential. Concrete shall flow to all parts of the structural forms uniformly and in sufficient quantity for which the appropriate arrangements of pouring, pinning, compaction and vibration shall be made. Needle vibrators shall be used for small depth wide section such as well staining, well-cap, piers, deck slab etc. but form vibrators shall be used when the structure to be concreted is thin and deep as in PSC girders. If such structure are vibrated by needle vibrators, they may cause damage to the prestressing sheaths or displacement of the non-tensioned reinforcement. Damage of sheath may permit ingress of cement slurry thereby jamming the prestressing cables. Displacement of non-tensioned reinforcement may reduce the cover resulting in corrosion for inadequate cover. Provision of inspection slits at suitable intervals at the bottom flange and web ensures that concrete is flowing to all parts.

24.2.9 Curing of Concrete

The hydration of cement requires the presence of moisture and as such proper curing of concrete structures by water is necessary after their casting. Generally, seven days' curing period is the minimum specified but longer period of curing is beneficial to the concrete structures. Curing of flat surface such as well cap, deck slab, wearing course etc. can be cured by ponding water but where ponding cannot be done such as vertical surfaces of well steining, pier & abutment shafts, pier cap, vertical face, bottom bulb etc. of concrete girders, curing may be done of wrapping gunny bags and sprinkling water over them. The water for curing shall be the same as used for mixing concrete.

24.2.10 Layout of Foundations

Before the start of the foundation work, layout of the foundation shall be correctly given and checked by independent technical personnel as any error in this respect will create problems in the construction of superstructure specially in precast/prefabricated girders. For land structures such as viaducts and overbridges, setting out of centre line of foundation by direct measurement is possible but for foundations in rivers or channels with water, such direct measurement is not possible and therefore indirect measurement as indicated in Fig. 24.1 shall be undertaken. A base line is laid on the river banks and on this base line, the centre line distances of foundations laid on ground such as 1′, 2′, 3′, 4′, etc. The actual centre lines of foundations in the river such as 1, 2, 3, 4, etc. over made-up ground by islanding, cofferdam etc. (Art. 21.5.11.2) are set out by the use of theodolite which is placed on each centre line on land and the respective centre line of actual

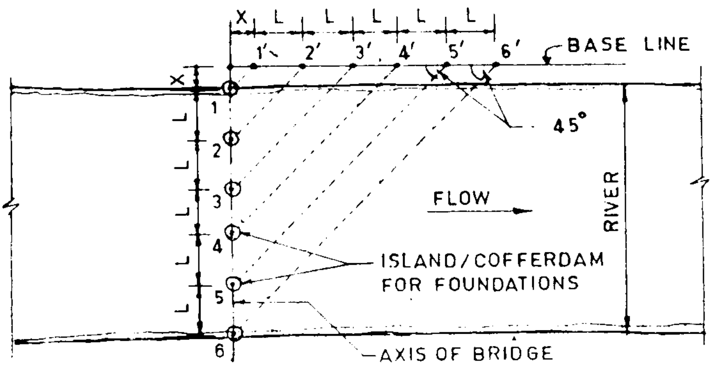


FIG. 24.1 LAYOUT OF FOUNDATIONS IN FLOWING RIVERS

foundation in the river is obtained by setting an angle of 45 degrees with the base line. The centre lines of the foundations so fixed in the rivers shall be checked by placing the theodoline on these centre lines and measuring back 45 degrees with the axis of the bridge to cut the centre lines on land.

24.2.11 Construction of Foundations

24.2.11.1 After the setting out of the centre lines of the foundations, the foundation work may be undertaken. For foundations on land, either open raft foundations or pile foundations are adopted. The details of raft foundations have been given in Art. 21.3.2. The base raft may be cast after doing form-work, laying reinforcement and then concreting which have already been dealt in this chapter. Generally, a mat concrete (1:4:8) of 75 mm thickness is provided below base raft for the facility of laying reinforcement. The pile foundations have been described in Art. 21.4.

24.2.11.2 For foundations in river bed having some depth of water, either islanding or cofferdam shall be done as mentioned in Art. 21.5.11.2. In very deep water, caissons may be floated, carried to the location and grounded as explained in Art. 21.5.11.2. Well or caisson sinking has been outlined in Art. 21.5.12. The works like form-work, laying of reinforcement, concreting etc. for well shall be as described in this chapter in Art. 24.2.6 to 24.2.9.

24.2.12 Construction of Substructure

24.2.12.1 The piers and abutments have been described in chapter 19. Their final siting on the pile or well cap shall be done after due consideration of the shift of C. G. of the pile group during driving or tilts and shifts of wells during sinking respectively.

24.2.13 Construction of Superstructure

24.2.13.1 Construction of solid slab, T-beam and slab etc. involves the form work, centering, laying of reinforcement etc. which have already been discussed in this chapter. Some special types of superstructure are described in the succeeding paragraphs.

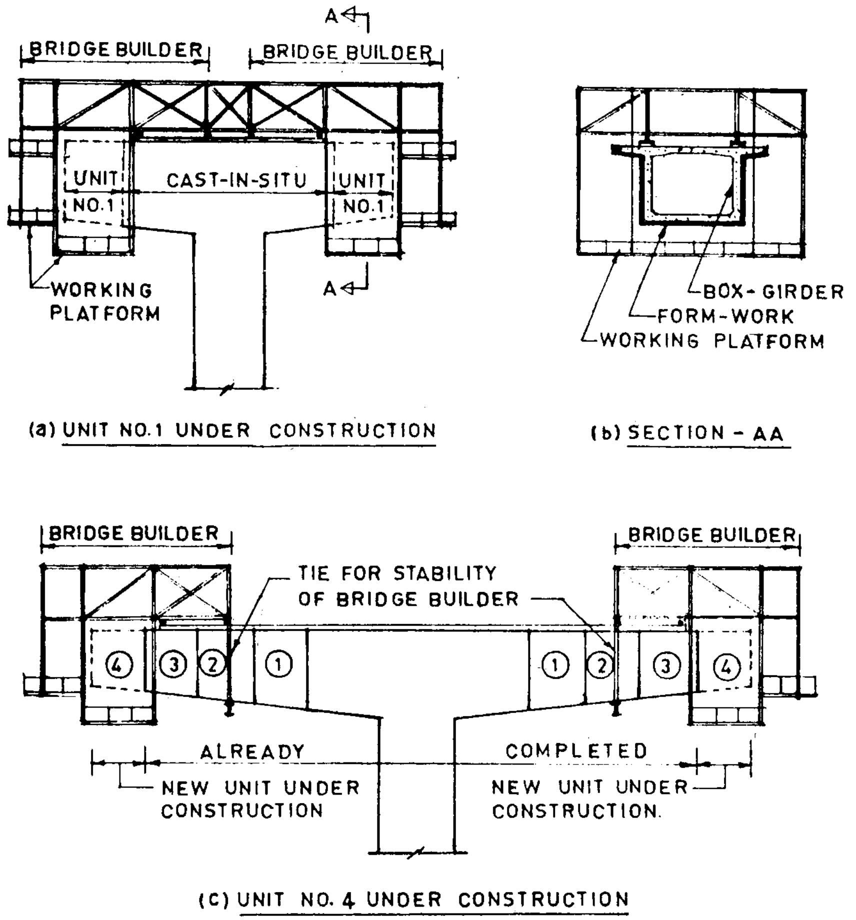


FIG. 24·2 CANTILEVER CONSTRUCTION (Jatana)[10]

24.2.13.2 *Box-girder Decks*

The soffit slab and ribs of R. C. hollow-box girder bridges are cast over form-work and centering from bed level. For casting deck slab, the form work is supported from the soffit slab. To remove these form-work, openings in the intermediate cross-girders and man-holes in the soffit slab are provided. Small holes (40 to 50 mm dia) are usually kept in the web at certain intervals for ventilation of the hollow boxes.

Prestressed concrete box-girder bridges are generally provided for moderately large spans where free-board is high from navigational consideration or where depth of water is considerable for which conventional centering is not possible. In such cases, cantilever construction method is adopted (Fig. 24.2). This method envisages the construction of the deck as a cantilever and hence is termed as ***"Cantilever construction"***. After casting of the pier head or hammer head (as it is sometimes termed due to its shape) by form-work and centering from the piers/well cap, stage prestressing is done.

Thereafter, the equipment of the cantilever construction which is called the bridge builder is placed on rail over the already cast deck, i.e., hammer head. The bridge builder is supported on wheels over rails and can be moved forward when required. Over the hammer-head, the two units of the bridge builder is inter- connected and therefore, maintain the stability while carrying the weight of the first unit cast on both sides symmetrically Fig. 24.2a. The form-work for soffit slab, sides of girders and deck slab are all suspended from the bridge builder. Even the platform for the working men during concreting and prestressing are hung from the bridge builders by suspenders. The concreting of the entire box-section, that is, the soffit slab, ribs and deck slab is done in one operation.

After casting of unit no. 1 on both sides with the help of bridge builder as shown in Fig. 24.2(a), stressing of the prestressing cables is done and anchored at the free end of unit No. 1 (for cable arrangement in cantilever construction, refer Fig. 16.25), when concrete attains the required strength. The form-work is released from unit no. 1 of both sides. For easy removal of the form-work, the bridge builders are supported over jacks and kept over them till the release of the shuttering. For release of shuttering, the jacks are removed and the bridge builder is placed on wheels over rail tracks placed on the already concreted deck. When shuttering is released, both the bridge builders, now on rails, are brought forward with the help of winch to the next unit no. 2 on either side simultaneously. The stability of the bridge builder against its own weight (including form- work, working platform etc. and including the weight of green concrete of the unit to be cast and working live load) is maintained by steel ties connecting the bridge builder at one end and R. S. J. placed below the soffit of already concreted unit. The form-work is fixed to unit no. 2 and m.s. reinforcement, cables etc. are placed in position and unit no. 2 on both sides is concreted simultaneously. This process, namely release of shuttering, shifting of the bridge builder to the next unit, fixing shuttering, m.s. reinforcement and H. T. Cables, concreting and prestressing is repeated for each unit or segment till the entire cantilever is cast on both sides. The stage prestressing done after casting each segment makes the cantilever already constructed strong enough to carry the design load at every stage. The method of construction is very quick avoiding the erection of centering and scaffolding or the use of launching truss. The bridge builder and the form-work attached to it can be used again and again and also in some other similar bridges for which the cost saving in form-work and centering is considerable although the initial cost of the bridge builder is high. Considering all aspects, cantilever construction for moderately large span bridges is very favourite now a days.

24.2.14 Construction of Arch Bridges

24.2.14.1 For R. C. arch bridges, the form-work and the centering are the most difficult operation because the arch bridges are mostly constructed over gorges where the bed level is very low and as such erection of centering from the bed is not possible. A number of methods are used for centering avoiding erection from the bed of the river. Some method are shown in Fig. 24.3.

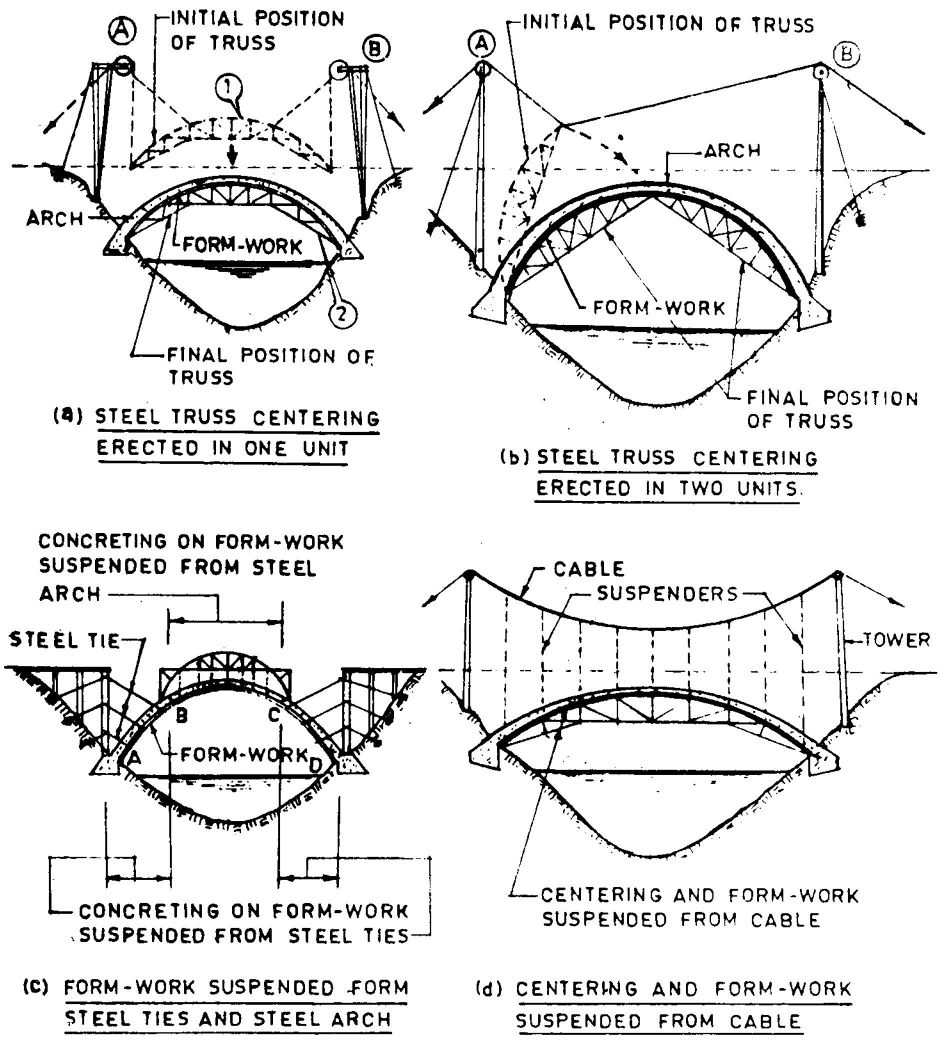


FIG.24.3 CENTERING AND FORM-WORK FOR ARCH BRIDGES

24.2.14.2 In Fig. 24.3(a), the prefabricated truss has the shape of the top chord coinciding with the soffit of the arch. The truss is brought to position 1 and then lowered to position 2 by means of two derricks A and B placed on either bank. In Fig. 24.3(b), the span of the arch being more, two separate arches are used and lowered by derricks A and B as shown. Fig. 24.3(c) shows the centering technique adopted in the construction of Coronation Bridge over Teesta River on N. H. 31 (West Bengal). The centering from A to B and C to D was supported by steel ties and the arch rib on these portions was cast. Thereafter, light steel arch fully fabricated was placed on the arch rib already cast and still supported by the ties. Then the concreting of the central portion of the arch rib was done on centering and form work suspended from the arch. The centering for the construction of the arch rib in Fig. 24.3(d), is suspended from the cables supported over towers and anchored into the ground as in a suspension bridge.

24.2.14.3 After the erection of form-work, the casting of the arch rib may be undertaken after laying of reinforcement. The reinforcement from abutments for fixed arches or the reinforcement for hinges at the springing for hinged arches (Fig. 13.19) shall be provided in the arched ribs before casting of the rib. The walls for the spandrel filled arches or the columns for the open spandrel arches may be constructed from the arch ribs and the deck is provided over these walls or columns. The deck may consist of deck slab over longitudinal beams and cross beams.

24.3 ERECTION OF BRIDGES

24.3.1 For erection of bridges either of concrete or of steel, various methods are deployed. Some of the method are now described for the erection of steel bridges and thereafter, for the erection of concrete bridges.

24.3.2 Erection of Steel Bridges

24.3.2.1 The methods of erection of some temporary/semi- permanent steel bridges such as Bailey or Callender-Hamilton have been illustrated in Fig. 18.4 and 18.6 (Art. 18.1.3.7 and 18.1.4.4). The same methods of erection are used in many permanent steel bridges.

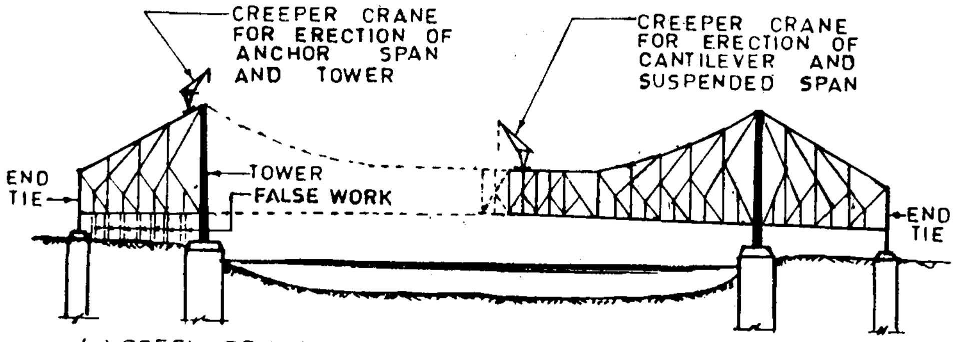
24.3.2.2 The erection of steel truss bridges may be done by using erection cranes. In Fig. 24.4(a), construction of Howrah Bridge, a cantilever truss bridge, is shown. The completed bridge has been described in Art. 17.6.3 and shown in Fig. 17.8. The erection of the anchor span (where there was no water) was done over false-work by creeper cranes. In the river portion, false- work was not possible as there was considerable depth of water and as such the truss starting from the tower up to the centre of the suspended span was erected by creeper cranes by cantilever method of construction. The anchor span held by the end tie provided the necessary stability for the cantilever construction. A temporary tie was also used at top chord level at the junction of the cantilever and suspended span for cantilever construction of the suspended span. By this process, the cantilever span as well as the half length of the suspended span were erected from both tower ends and the central gap was closed. Thereafter, the suspenders were hung from the nodal points of the bottom chord of the truss and the deck was constructed over longitudinal girders and cross-girders supported on suspenders.

Fig. 24.4(b) shows the construction of a simply supported truss bridge also by cantilever construction method but here two derrick cranes are simultaneously used on both sides of the pier and the construction proceeds towards centre of span symmetrically from stability consideration. A temporary tie is used over the pier at top chord level for this purpose. To construct some portion of the truss over the pier in order to have a platform for the two working cranes, temporary struts at bottom chords are used from wells or piers. This method is similar to the cantilever construction of PSC bridges as shown in Fig. 24.2.

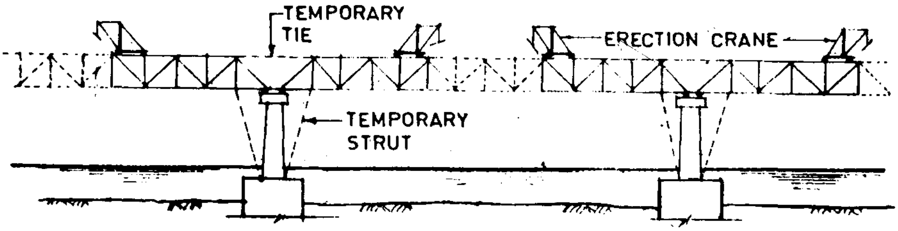
24.3.2.3 Steel arch bridges in deep gorges (or in situations where centering from bottom for erection purpose is not possible) may be constructed by special cable hoisting system as shown in Fig. 24.4(c). A rope may be hoisted over temporary towers with the help of helicopter. Further strengthening of the rope may be done by spinning additional wires as in a suspension bridge as described in the next paragraph. The components of the arches may, therefore, be carried through this hoisted cable and the arch bridge is constructed. This erection technique was adopted in the steel arch bridge 500 m. long over the New River Gorge, near Fayetteville, West Virginia, USA.

24.3.2.4 The suspension bridge erection as shown in Fig. 24.4(d) consists of the following stages :

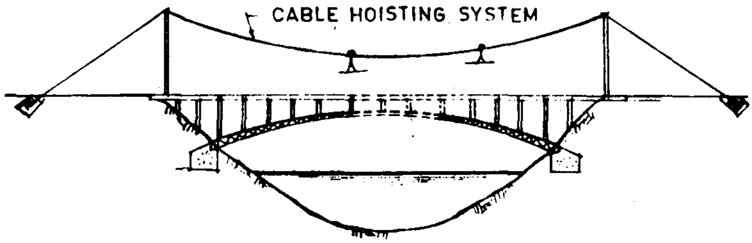
(i) Erection of towers and anchorages
(ii) Providing cat walk
(iii) Spinning of the main cables and fixing them with the anchorages and towers.
(iv) Erection of suspenders and stiffening truss
(v) Construction of flooring system.



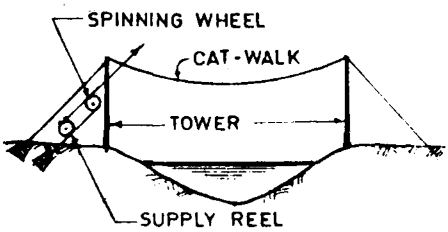
(a) STEEL TRUSS BRIDGE – PART ERECTION ON FALSEWORK AND PART ERECTION BY CANTILEVER CONSTRUCTION



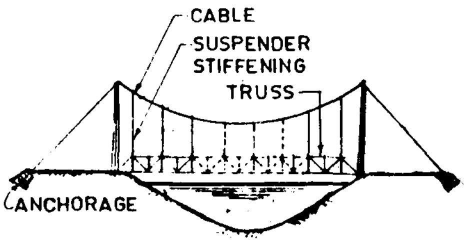
(b) STEEL TRUSS BRIDGE - ERECTION BY CANTILEVER METHOD



(c) CONSTRUCTION OF STEEL ARCH BRIDGE BY CABLE HOISTING SYSTEM



(i) AERIAL SPINNING OF CABLE



(ii) ERECTION OF SUSPENDERS & STIFFENING TRUSS

(d) CONSTRUCTION OF SUSPENSION BRIDGE

FIG. 24·4 ERECTION OF STEEL BRIDGES (Smith)[18], (Victor)[21], (IRC)[6]

After erection of the towers and completion of the anchorage-arrangement, a catwalk is provided with timber platform over ropes which are placed concentric with main cables. A tramway system is installed over the cat-walk for spinning the wires for the cables. At each anchorage, a spinning wheel is attached tot he tramway system. The spinning of the wires (which is known as *"aerial spinning"*) is done by fastening the ends with the anchorages and then making loops over the spinning wheels. The spinning wheels are pulled along the catwalk and over the towers to the opposite anchorages. The wires are then attached to the anchorages and this procedure is repeated till all the wires of the strand are carried over the towers to the anchorages. The entire strand is then banded up at intermediate places. After completing all the strands of the cables in the way as described, the cable is compacted by squeezing to the form of a circular section.

24.3.3 Erection of Concrete Bridges

24.3.3.1 Erection of concrete bridge generally means erection of prestressed concrete bridges as erection of reinforced concrete bridges is rarely done. However, one reinforced concrete arch bridge was erected in Japan by means of a new construction method unprecedented in the world as claimed. This is the Hokawazu Bridge over Hokawazu Creek between Chinzei and Genkai at Higashi-Matsuura County. The bridge has a central span of 170 m with total length of 252 m and is the longest R. C. arch bridge in Japan with floor level at 50 m above sea level. The method of construction is described below.

24.3.3.2 A cantilever construction method was adopted in this bridge in which the segments formed of an arch rib, struts and floor slab were supported by prestressing steel bars and the overhanged bodies extended their length in stages from both the banks towards the centre until the last segment is placed at the centre (Fig. 24.5a).

24.3.3.3 Erection of PSC beams can be done by the use of gantry as shown in Fig. 24.5(b). This method is suitable for land spans or in river bed, where the dry weather flow is small and is limited to a very small width of the bed. The height of erection is about 10 metres.

24.3.3.4 The erection of PSC beams in the approach viaduct of the second Hooghly Bridge, Calcutta was done by the use of tilting derricks as shown in Fig. 24.5c. Two derricks were used, one at each and of the girder, to lift the girder over the pier. These derricks were then tilted by releasing one of the guy ropes and tightening the other very slowly and carefully keeping both the guy ropes taut. The girder was then placed over the pier cap and side-shifted to its actual position by usual process.

24.3.3.5 In deep water rivers where normal staging is not possible, the erection of girders may be done by the use of launching truss. Fig. 24.6a shows such a scheme adopted for the construction of Rupnarayan Bridge at Kolaghat on N. H. 6 (West Bengal). The prestressed concrete girders, 46.0 metres in length between centre line of bearings were cast and stressed on the approaches, placed over two trolleys at two ends. The trolleys were than run over rail lines and the girders were brought near the abutments where a launching truss as shown in Fig. 24.6a was standing. Both ends of the girder were lifted from the trolley simultaneously and suspended from the bottom boom of the launching truss. The suspenders had a pair of wheel at top resting on the bottom boom through which the girders could be moved longitudinally. In this way, the girders were brought over the first span and lowered one by one by the use of sand jacks and side-shifted to their actual position. After the first span girders were launched, the rail track was extended over the already launched girders and the launching truss having a balancing tail truss with water tanks at the end was moved to the next span. The balancing truss maintained the stability of the launching truss during its shifting to the next span. After the launching truss was moved to the next span and fixed, the process of launching the girders carried from the casting yard was repeated as before till all girders of all spans were launched and side shifted in position.

24.3.3.6 Argentina's Chaco-corrienties Bridge linking the coastal part of the country with the Western plains is a cable- stayed bridge using precast concrete box-girder sections 3.5 m × 2.5 m forming the bridge deck (Fig. 24.6b). The portion of the deck between the inclined struts B to C was cast in place to provide a platform

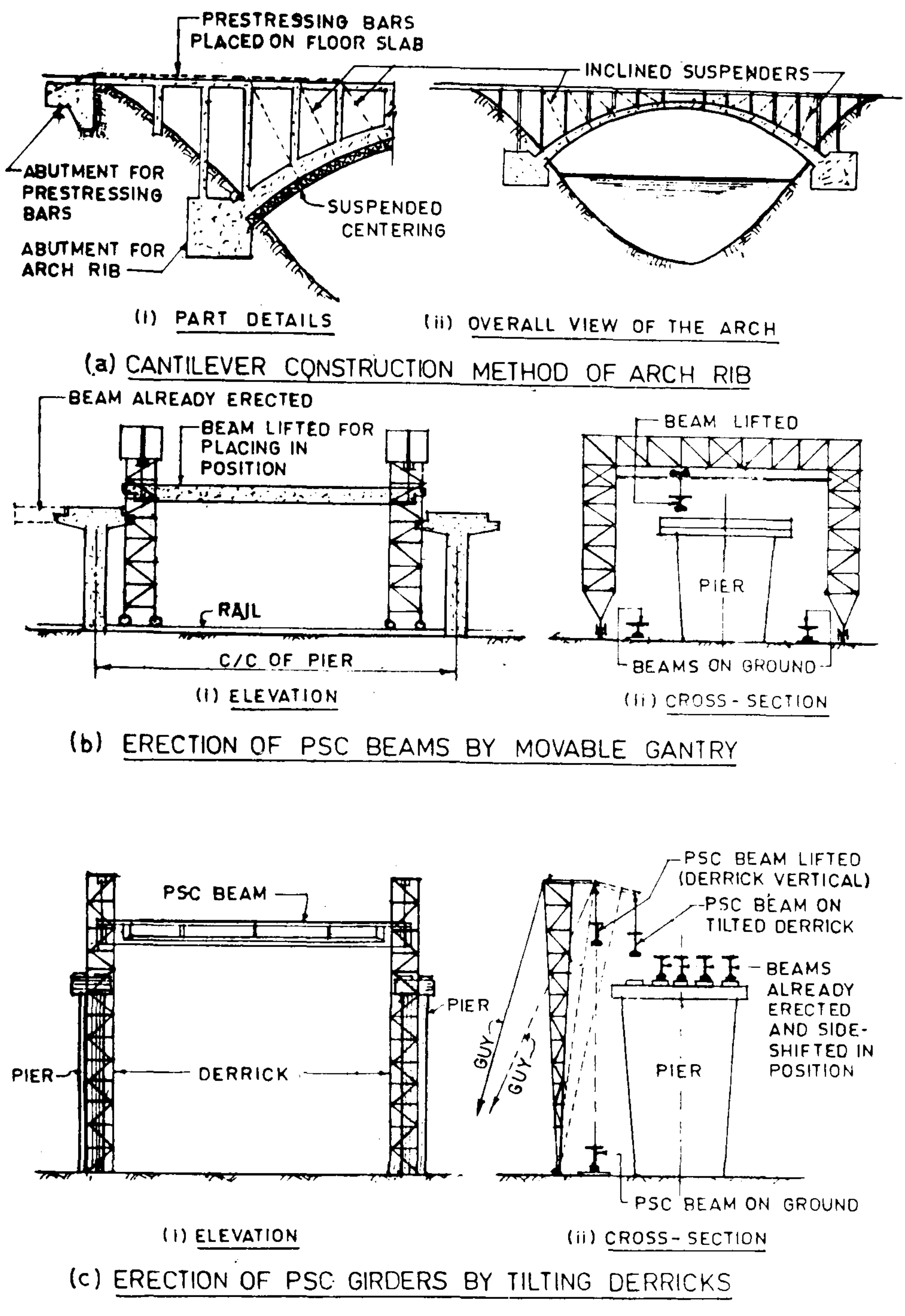
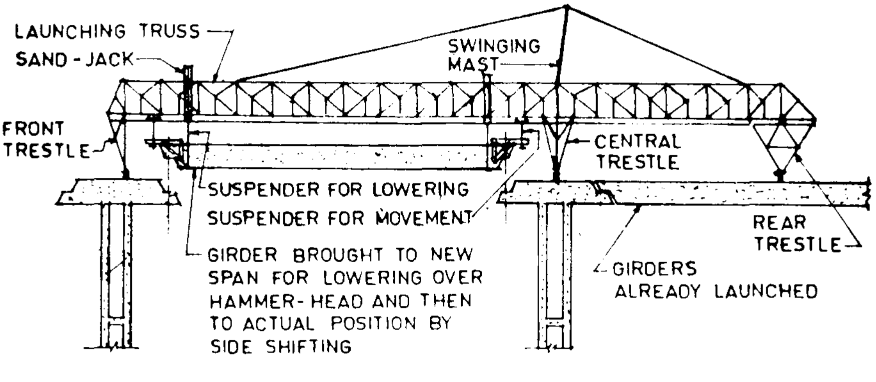
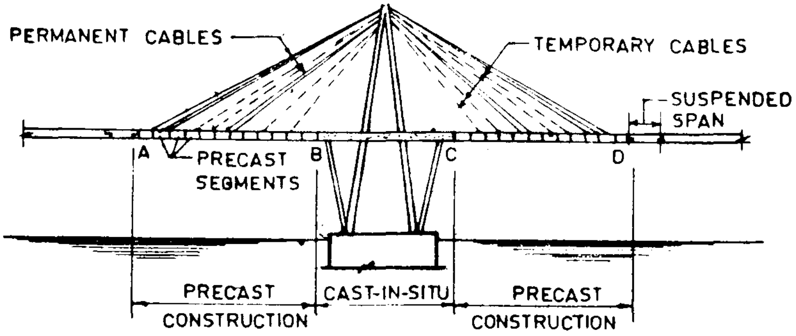


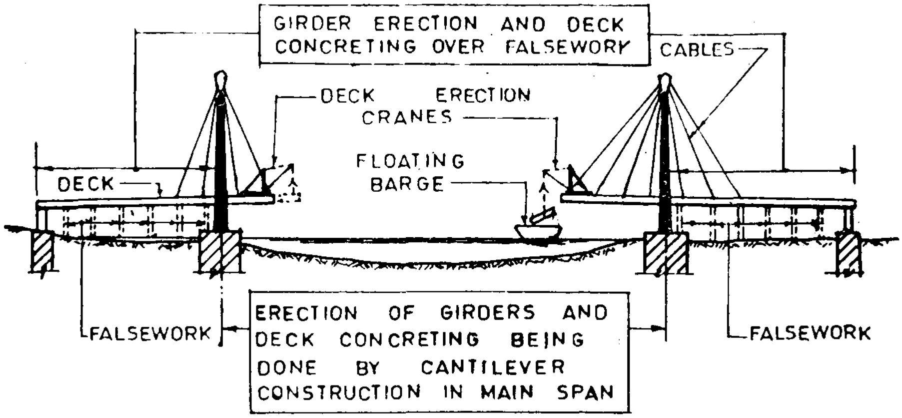
FIG. 24.5 ERECTION OF CONCRETE BRIDGES (IRC)[4] (Apte et al)[7]



(a) ERECTION OF PSC GIRDERS BY USING LAUNCHING TRUSS



(b) ERECTION OF PRECAST CONCRETE BOX SECTIONS BY USING CABLE STAYS



(c) ERECTION OF STEEL-CONCRETE COMPOSITE DECK BY CANTILEVER CONSTRUCTION USING PERMANENT CABLE STAYS

FIG. 24.6 ERECTION OF CONCRETE / COMPOSITE GIRDERS

(Apte et al)[7], (Rothman et al)[15], (Rakshit)[14] for the erection of the precast box-girder sections in the portions A to B and C to D. The precast box-girder sections were cast in casting yard against each other for proper matching. The segments were floated to the site by barges, lifted by a ua.ening crane, placed on top of the already completed deck, placed against the previously erected deck and prestressed. Each unit of the bridge deck, i.e., C to D is supported by two sets of stay cables in each cantilever span, i.e., eight sets for the entire unit considering both towers. In addition, four sets of temporary cables, though of smaller strands, had to be used in each cantilever from each tower for the facility of erection by cantilever construction method.

24.3.3.7 The Second Hooghly Bridge now under construction at Calcutta is a cable-stayed bridge the details of which have been given in Art. 17.7.10 and the general arrangement of the bridge has been shown in Fig. 17.17. Similar to Howrah Bridge, the shore spans have been completed over false-work in this bridge also (Fig. 24.6c). Steel towers were erected by tower erection cranes which moved vertically upwards along the towers as the latter's erection proceeded. Deck erection cranes lifted the steel main and cross-girders from below and placed and fixed them in position over false-work for the shore spans. Thereafter, the concrete deck slab was cast while the shore spans were still supported over false-work. After the completion of the towers and the shore spans, two pairs of inclined cables are fixed from the tower to the shore and the main span simultaneously starting from tower side cables. While the cables are fixed to the shore spans which have been completed over false-work, the main span cables support only the main and cross-girders of one panel length which is lifted from the floating barges by the deck erection cranes placed over completed deck. This panel length is then completed by casting the deck slab. Thereafter, the deck erection crane is moved forward towards centre, next unit of the main and cross- girders is lifted and fixed in position being supported by the next set of inclined cables from the tower and the deck slab is cast. In this process, the entire main span is being completed by cantilever construction method using inclined permanent cables as supports and proceeding simultaneously from both the towers towards the centre. As the main span cables increase in number along with the construction of the deck, the shore span cables are also added to bring the stability of the whole structural system.

24.4 MAINTENANCE OF BRIDGES

24.4.1 The maintenance of bridge means the upkeeping of the bridge components in good and serviceable condition so as to ensure a longer life of the bridge as envisaged at the time of its design and construction. Even if the bridges are well designed and properly constructed, periodic maintenance, if needed, is very essential to keep them in good serviceable condition. Therefore, the bridges should be regularly inspected and properly maintained.

24.4.2 The matters which require immediate attention and maintenance may be broadly stated as below :

(i) Approaches, (ii) Protective Works, (iii) Foundation, (iv) Piers, abutments and wing walls, (v) Bearings, (vi) Superstructure, (vii) Expansion Joints, (viii) Wearing Coat, (ix) Drainage Spouts, (x) Hand Rails, (xi) Foot Paths, (xii) Utility Services.

A proforma for Inspection Report for Road Bridges (Extract from Indian Road Congress *"Special Publication 18, Manual for Highway Bridge Maintenance Inspection, 1978"*) is given in Appendix-D.

24.4.3 Classification of Bridge Maintenance

24.4.3.1 The bridge maintenance may be broadly classified into two types, viz.

(a) Routine maintenance or annual maintenance

(b) Quarterly maintenance

Routine maintenance is the annual maintenance done on the basis of routine inspection and includes the following :

(i) For concrete structures-protection against exposed reinforcement and repairs to cracks.
(ii) Maintenance of bearings and expansion joints.
(iii) Maintenance of wearing coat.
(iv) Maintenance of kerbs, railing etc.
(v) Maintenance of weep holes & drainage spouts
(vi) Maintenance of protective works.

Quarterly maintenance based on detailed inspection shall cover the following items of works and shall be undertaken once in four years :

(i) Major repair to piers, abutments, wing walls.
(ii) Major repairs to guide bunds and bridge protection works.
(iii) Major repairs to the superstructure such as wearing course, railing, footpath slab, deck slab, girders etc.

24.4.3.2 In addition to the routine maintenance and quarterly maintenance, another sort of maintenance which may be termed as *"Special Repairs"* shall be undertaken as and when necessary. This group covers the following :

(i) Replacement of the bearings by lifting the superstructure.
(ii) Repairs to the articulation by lifting the suspended span.
(iii) For steel bridges, replacement of cross-girders and deckings, rivetting etc.

24.4.4 Mobile Bridge Inspection Unit

24.4.4.1 As stated in previous paragraphs, maintenance works are undertaken on the basis of the report of inspections but if such inspections can not cover all the bridge components, proper inspection and as such proper maintenance of bridges is not possible. This is specially so when the height of the bridge is considerable or there is always standing water under the bridge in which cases in-depth inspection from distance is impossible. Mobile Bridge Inspection Unit which could be able to carry the bridge inspecting officers close to every components of the bridge specially at the sides and the underside of the bridge is the solution to such problems.

24.4.4.2 The Public Works Department, Govt. of Maharashtra, has procured one such Bridge Inspection Unit from Finland. The model selected is Bronto skylift 13/10-4 which has the maximum vertical working range of 13 metres and the maximum horizontal working range of 10 meters of the inspection cage. The Mobile Bridge Inspection Unit (Shown in Fig. 24.7) is mounted on a Mercedez Benz 3 axle truck chassis mounted with six cylinder diesel engine. The system consists of four hydraulic outriggers controlled from the rear end of the vehicles. During the operation of the machine, more than half of the road is completely free for the movement of the existing traffic.

24.4.5 Types of Work Possible with the Bridge Inspection Unit (Bronto Skylift 13/10-4)

24.4.5.1 It has been reported that the following inspection and repair works are possible with Bronto Skylift 13/10-4 :

(i) Each and every component of the bridge under side, bottom of slab, sides of girders, top of pier etc. can be inspected very closely.
(ii) Greasing of and repairs to the bearings.
(iii) Patch repairs to damaged/cracked concrete of slab, girder, pier cap etc.
(iv) Painting of underside of steel bridge deck and all sides of steel girders.
(v) Snowcem or similar cement works of underside of deck and all sides of girder of concrete bridges.
(vi) Measures for the prevention of corrosion can be taken.

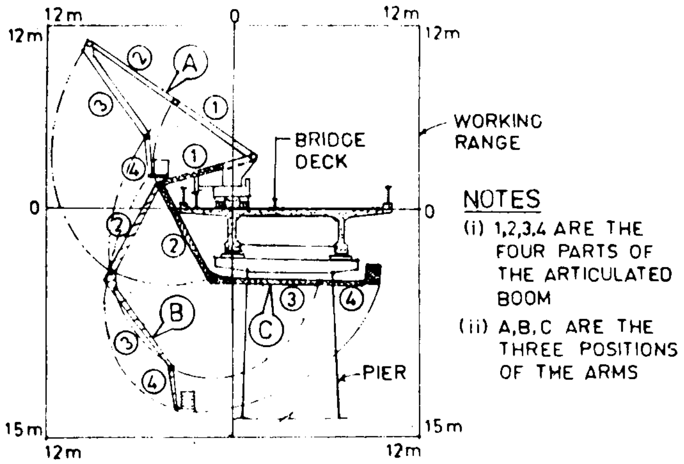


FIG. 24.7 MOBILE BRIDGE INSPECTION UNIT
(BRONTO SKY-LIFT 13/10-4) (Naik)[11]

(vii) The Bronto Skylift is fitted with pneumatic pressure hose system and as such guniting, spray painting, sand blasting or cleaning of concrete surface with air pressure can be done.

It is, therefore, absolutely necessary to possess such or similar type of mobile Bridge Inspection Unit for the sake of close and in-depths inspection of all components, specially the underside and sides of the bridges. This is also necessary to reach the otherwise in-accessible portions of the bridge for carrying out repair works from a close distance. If proper maintenance of the bridges can be done, better serviceability and longer life of the bridges can be expected.

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CHAPTER 25

BRIDGE ARCHITECTURE

25.1 GENERAL

25.1.1 The bridge designer of this age has to be not only an engineer but also an artist. Along with the functional and structural design, the bridge designer shall add strong feeling for beauty of form, line and proportion.

25.1.2 The engineer may consider it important to design a bridge using the most advanced theorem and technology but for a public, the aesthatic consideration of the bridge is the most important as they do not bother about the design but about the appearance. The form, line, proportion, texture and colour of a bridge should be pleasing and beautiful to him. It is, therefore, imperative that a bridge should not only be safe and sound or ideal from the consideration of safety and economy but also it should appear beautifully. A bridge designer should, therefore, keep in view this aspect while designing the bridge so that the aesthatic appearance of the bridge is improved. This may be achieved in general, if the treatment to the vissible parts of the bridge in respect of mass, line, colour, texture etc. is made harmoniously. The engineer has to match bridge structure with the surroundings such that the resulting effect can make a lasting impression in the minds of everybody.

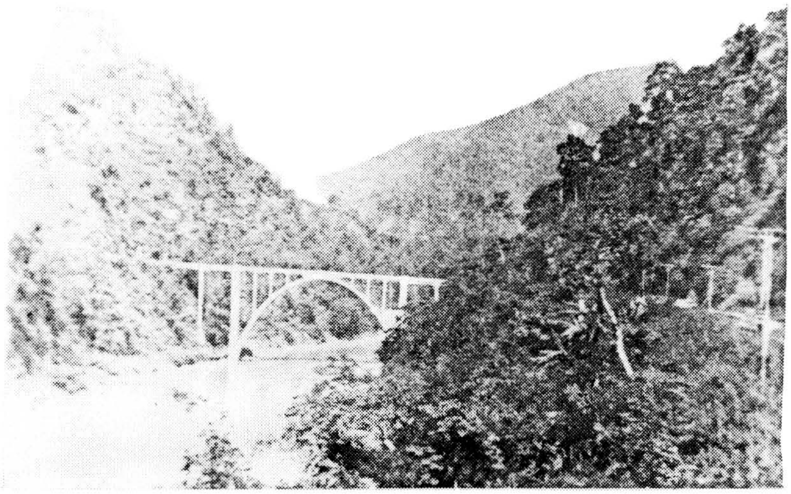
25.2 FEW BASIC POINTS OF BRIDGE ARCHITECTURE

25.2.1 Aesthatics is a matter of taste and, therefore, it is not possible to codify the rules which are to be followed in the design of bridges with a view to getting the desired result in respect of the bridge architecture. However, a few basic criteria for general guidance are mentioned below.

25.2.2 Blending the Structure with the Landscape

25.2.2.1 The bridge should blend with the landscape or the environment. In a narrow gorge with a beautiful background, a deck type arch bridge specially with a high rise- span ratio is most suitable (Photograph 6).

25.2.2.2 In an area where stones are available in abundance and where stone out-crops are vissible in the locality, a bare stone arch bridge will beautifully match the surroundings. Heavy treatment in the design of arches, abutments or parapets is generally more pleasing in rugged country than in a flat country. Masonry or mass concrete is normally used as chief construction materials for bridges in the former case whereas lighter materials such as reinforced concrete or prestressed concrete is used in the latter case.



Photograph 6
CORONATION BRIDGE OVER TEESTA RIVER
(R.C. Arch Bridge on N. H. 31 in the State of West Bengal having a clear span of 82.0 m and a rise of 14.0 m) *(Highway Bridges)*[2]

25.2.3 Integration of the Road and the Bridge

25.2.3.1 A good aesthatics demands the integration of the road and the bridge as a whole so that a common goal, namely, a pleasing and harmonious transportation system is achieved. Since the road and the bridge are designed by two different groups of engineers, a liaison between the two is absolutely essential to have an aesthatically pleasing transportation system.

25.2.4 Proportioning of Various Components

25.1.6.1 The proportioning of various components of the bridge specially in relation with its environment is very important in achieving a good aesthetic appearance of the bridge. The depth of the structure must have some pleasing relation with the span although this again depends upon the type of structure adopted. A long span deck leaping over a valley may look beautiful (Fig. 25.1) but the same in a grade separated crossing will not be aesthatically pleasing. In the latter case, the bridge spanning between two abutments will have more

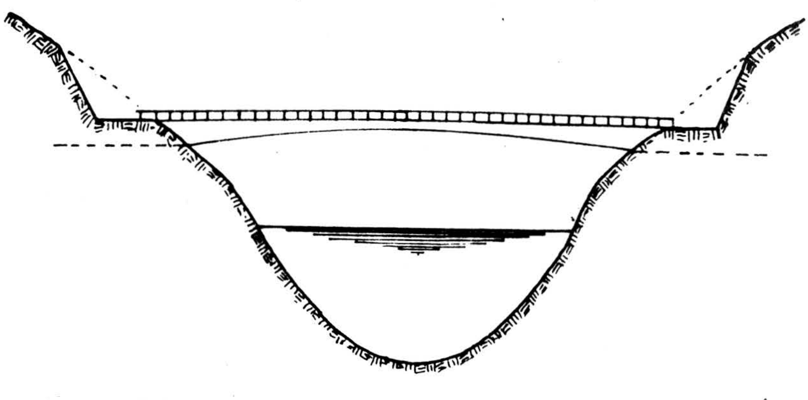


FIG. 25·1 LUBHA BRIDGE (ASSAM) (Gammon)[4] deck thickness and in an underpass with a headroom of 5.0 m or so, this bridge will be far less aesthatically pleasing.

25.2.5 Uniformity in the System of the Structure

25.2.5.1 The system of the bridge structure should be the same throughout. Mixing of systems in the same structure such as use of beams and arches or straight and curved soffit is not desirable.

25.2.6 Least Obstruction of View

25.2.6.1 The viaducts or overbridges across the expressway should be planned in such a way that obstruction of the view is the least. For viaducts, the piers should be as slender as possible (if required with more reinforcement) while for the overbridge, the provision of a long central span with two small end spans supported over two slender piers and two short abutments having a pitched slope in front is preferred (Fig. 25.2). Such overbridges with deep beams, thick piers or heavy abutment have the impression of narrow openings and, therefore, disturb the driver psychologically and affect his feeling of openness.

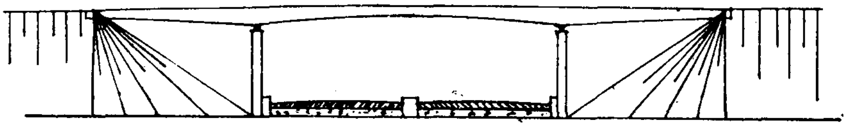


FIG. 25·2 THREE SPAN (CONTINUOUS) OVERBRIDGE

25.2.7 Treatment of the Approaches

25.2.7.1 The approach gradients either for a valley curve or for a summit curve shall have a continuity without a break. The change of slope should be gradual without any abrupt or sudden change. The approach to the bridge with a kink as shown in Fig. 25.3(a)(i) and 25.3(b)(i) shall be avoided. The approach gradients with two vertical curves at two ends with the bridge deck straight as shown in Fig. 25.3(a)(ii) and 25.3(b)(ii) is preferable to the former but a curved bridge deck in the vertical plane having a single, curvature as shown in Fig. 25.3(a)(iii) and 25.3(b)(iii) is still better.

25.2.7.2 The appearance of the approaches may be improved if the embankment or the cutting of the approaches near the entrance to the bridge is cleared off from all sorts of unsightly materials and the side slopes are properly maintained with grass turfing or pitching as found convenient.

25.2.8 Treatment of the Piers

25.2.8.1 The elevation of the pier looks better if cut and ease water is introduced in place of a square or blunt face. Similarly, if the pier base is made slender both in longitudinal and transverse directions as shown in Fig. 25.4, the elevation appears pleasing to the eye. The pier tops are sometimes moulded to conceal the bearing when seen in elevation (Fig. 25.4).

25.2.9 Shape of the Superstructure

25.2.9.1 In the early days when steel superstructure was mainly used for bridge construction, there was very little scope for bridge architecture except by adopting long majestic structures but with the advent of reinforced concrete and the easeness of providing lines and curves in the bridge shape, almost an unlimited scope for introducing varieties in the architectural treatment of the bridge structure was opened up.

25.2.9.2 Experience has shown that the deck with some curved soffit as in the case of continuous or balanced cantilever bridges given better aesthatic appearance in elevation than the deck with straight soffit. The

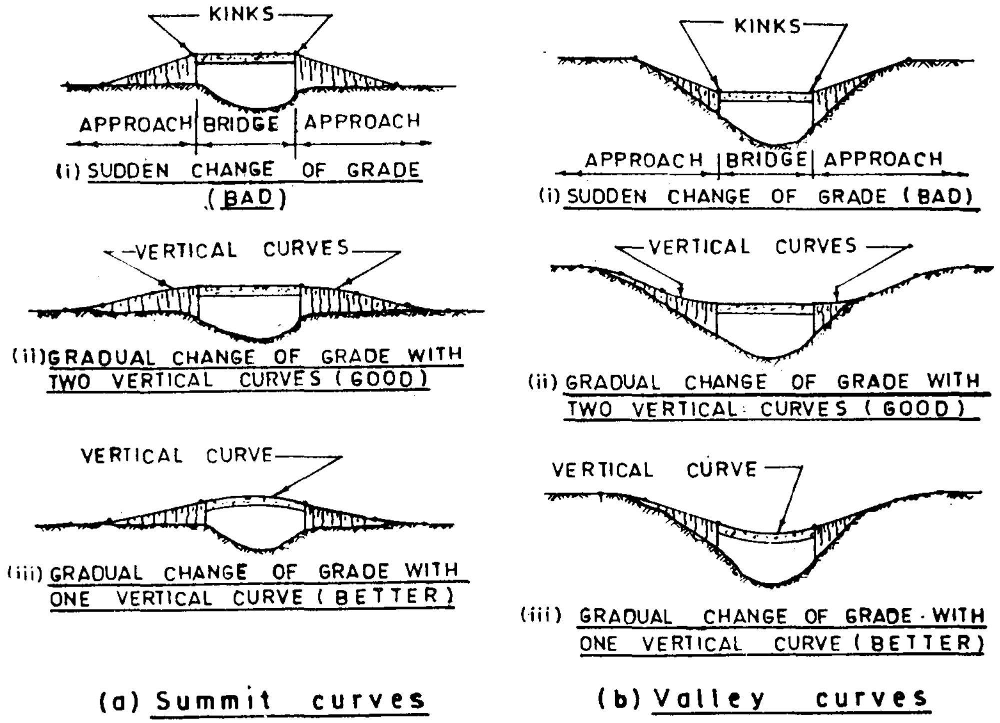


FIG. 25·3 BRIDGES WITH SUMMIT AND VALLEY CURVES

appearance can further be improved if in addition to the curved soffit, a vertical curve is introduced at the top of the deck (Fig. 25.5).

25.2.10 Moulding

25.2.10.1 Moulding of the components of the superstructure would enhance the appearance as these introduce the light and shade effect in the superstructure. Such mouldings may be in the form of coping, groove etc. (Fig. 25.6).

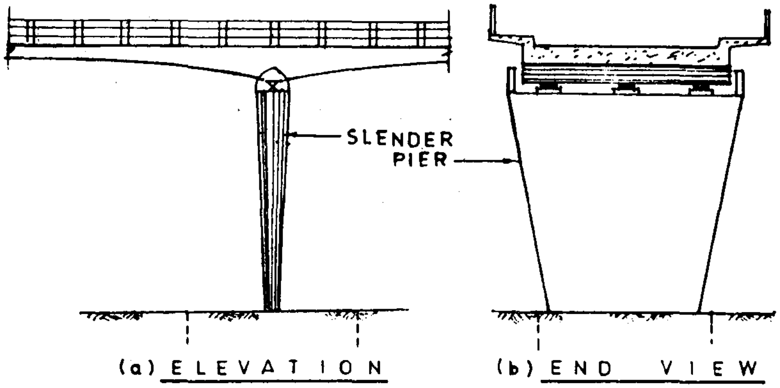


FIG. 25·4 TREATMENT OF PIERS

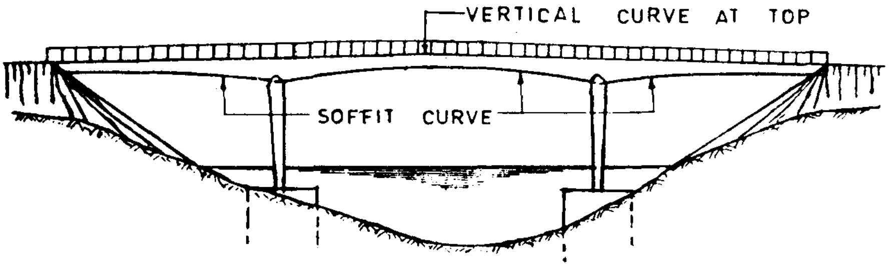


FIG.25·5 CURVED SUPERSTRUCTURE

25.2.10.2 Sometimes light posts are provided in the bridges for bridge lighting specially in urban areas. These light posts may be located just outside the railings on a specially designed platform with suitably moulded base. This will enhance the béauty of the bridge when seen in elevation (Fig. 25.6).

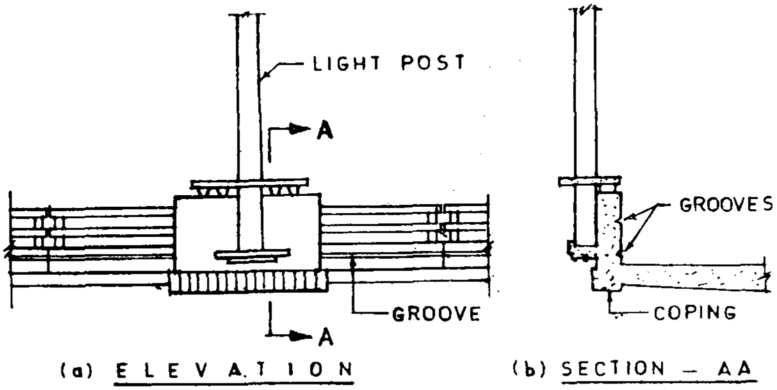


FIG.25·6 MOULDING IN BRIDGES

25.2.11 Parapets, Railings etc.

25.2.11.1 Heavy treatment in the design of parapets, railings etc. is desirable in rugged countries whereas simple, light parapets or railings are desirable in bridges built in plains. Solid parapets with some treatment say flutted, reeded, scrubbed or bush-hammered (Fig. 25.7) look better in spandrel filled arches. Open type railings are preferable in open spandrel arches, slab bridges or slab and girder bridges.

25.2.11.2 Solid parapets may be used at the entry of the bridge just ahead of the pylons even in R. C. or PSC bridges in plains where open railings are used in the main bridge portion (Fig. 25.8 and Photograph 7).

25.2.12 Pylons

25.2.12.1 Provision of pylons at the entrance to the bridge improves the architectural appearance of the bridge in elevation. The shape of the pylon and the motifs depicted on them are sometimes associated with the important personalities who lived nearby or with the culture or profession of the people living in the area so as to convey through these pylons the idea that the bridges are built for the people living in the locality. Sri Ramkrishna Setu over river Dwarakeswar at Arambagh (West Bengal) is situated near Kamarpukur, the birth-place of Sri Ramkrishna Paramhansa. The pylon of this bridge is a replica of the humble cottage in which Sri Ramkrishna lived while at Kamarpukur (Fig. 25.8).

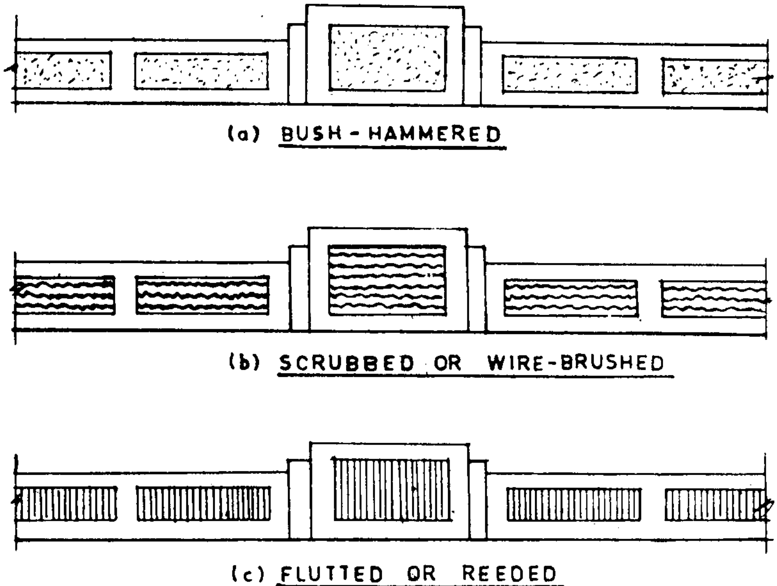


FIG. 25·7 TREATMENT ON PARAPETS

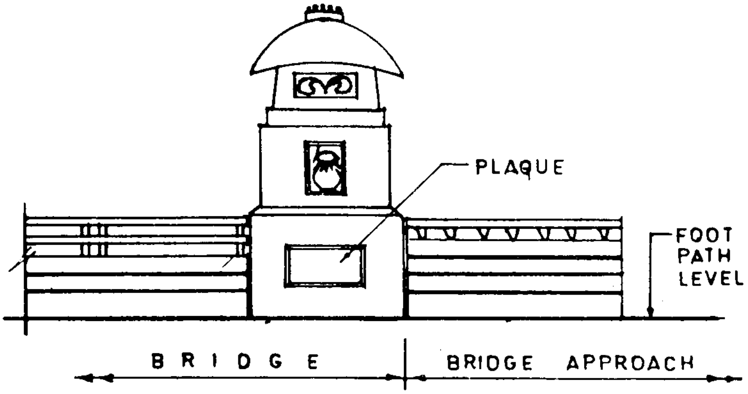
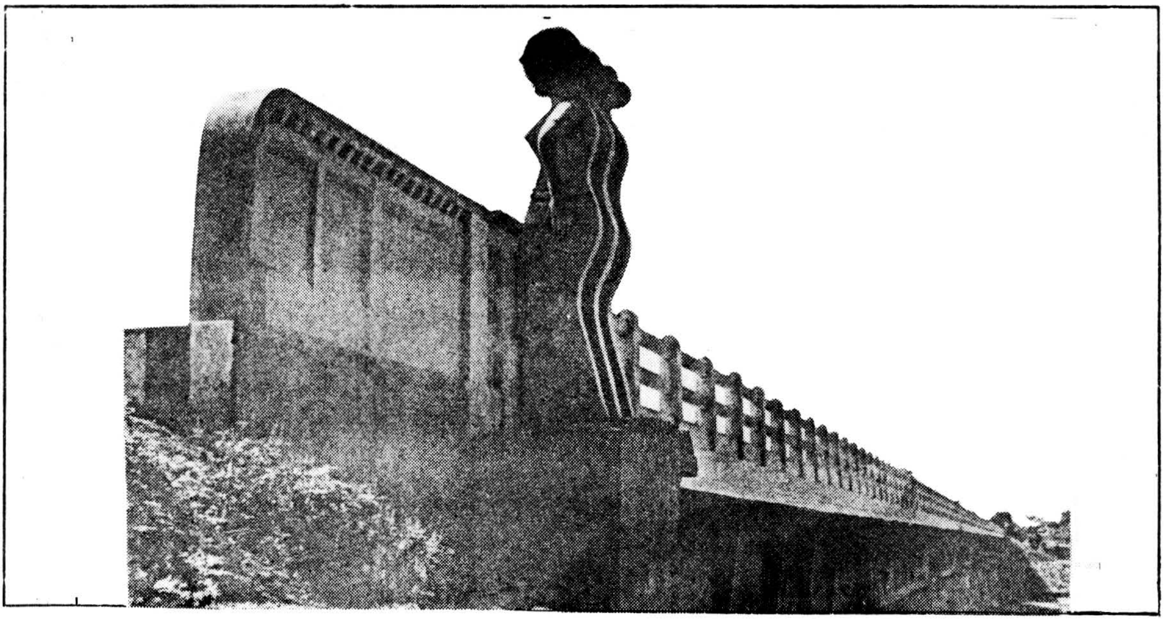


FIG. 25·8 PYLON OF SRIRAMKRISHNA SETU (Highway Bridges)[2]

25.2.12.2 The bridge over river Kangsabati at Raipur in the district of Bankura (West Bengal), is located in the land of the santhal tribes. The pylon of the bridge depicts three santhal women attired in their traditional garments in the posture of welcoming the visitors and as such symbolises the land and the people (Photograph 7).

25.2.13 External Finish or Treatment

25.2.13.1 The external surface of concrete bridges require rubbing with carborundum stones and water to have a good smooth surface finish. Bridges are sometimes colour-washed or snowcem washed or painted to enhance the beauty of the bridge. However, the colour finish shall be done in such a way that it matches the surroundings of the locality.



Photograph 7
PYLON OF KANGSASABATI BRIDGE AT RAIPUR IN THE DISTRICT OF BANKURA, WEST BENGAL
(Highway Bridges)[2]

25.3 REFERENCES

1. Allan, B. J. — *"Aesthatics in Bridge Engineering"* — Transport Communication Monthly Review (February & March, 1973), Indian Roads Congress.
2. Booklet on *"Long-span Bridges and Long-span Roof Structures"* — Gammon India Ltd., Gammon House, Veer Savarkar Marg, Prabhadevi, Bombay - 25.
3. First International Symposium — *"Concrete Bridge Design"* — ACI Publication SP – 23.
4. *"Highway Bridges in West Bengal"* — Public Works and Public Works (Roads) Departments, Govt. of West Bengal, 1977.

Appendix – A

HYPOTHETICAL VEHICLES FOR CLASSIFICATION OF VEHICLES AND BRIDGES (REVISED)
NOTES FOR LOAD CLASSIFICATION CHART

1. The possible variations in the wheel spacings and tyre sizes, for the heaviest single axles — cols. (f) and (h), the heaviest bogie axles — col. (i) and also for the heaviest axles of the train vehicle of cols. (e) and (g) are given in cols. (j), (k), (l) and (m). The same pattern of wheel arrangement may be assumed for all axles of the wheel train shown in cols. (e) and (g) as for the heaviest axles. The overall width of tyre in mm may be taken as equal to [150 + (p–1) 57], where "p" represents the load on tyre in tonnes, wherever the tyre sizes are not specified on the chart.

2. Contact areas of tyres on the deck may be obtained from the corresponding tyre loads, max. tyre pressures, col. (o) and width of tyre treads.

3. The first dimension of tyre size refers to the overall width of tyre and second dimension to the rim diameter of the tyre. Tyre tread width may be taken as overall tyre width minus 25 mm for tyres up to 225 mm width, and minus 50 mm for tyres over 225 mm width.

4. The spacing between successive vehicles will be 30 m. This spacing will be measured from the rear-most point of ground contact of the leading vehicles to the forward-most point of ground contact of the following vehicle in case of tracked vehicles ; for wheeled vehicles, it is measured from the centre of the rear-most axle of the leading vehicle to the centre of the first axle of the following vehicle.

5. The classification of the bridge shall be determined by the safe load carrying capacity of the weakest of all the structural members including the main girders, stringers (or road bearers), the decking, cross bearers (or transome) bearings, piers and abutments, investigated under the track, wheel axle and bogie loads shown for the various classes. Any bridge up to and including class 40 will be marked with a single class number — the highest tracked or wheel standard load class which the bridge can safely withstand. Any bridge over class 40 will be marked with a single class number if the wheeled and tracked classes are the same, and with dual classification sign showing both T and W load classes if the T and W classes are different.

6. The calculations determining the safe load carrying capacity shall also allow for the effects due to impact, wind pressure, longitudinal forces, etc. as described in the relevant clauses of this code.

7. The distribution of load between the main girders of a bridge is not necessarily equal, and shall be assessed from considerations of the spacing of the main girders, their torsional stiffness, flexibility of the cross-bearers, the width of roadway and the width of the vehicles, etc. by any rational method of calculations

8. The maximum single axle loads shown in columns (f) and (h) and the bogie axle loads shown in column (i) correspond to the heaviest axles of the trains, shown in columns (e) and (g) in load-classes up to and including class 30-R. In the case of higher load classes, the single axle loads and bogie axle loads shall be assumed to belong to some other hypothetical vehicles and their effects worked out separately on the components of bridge deck.

9. The minimum clearance between the road face of the kerb and the outer edge of wheel or track for any of the hypothetical vehicles shall be the same as for Class AA vehicles, when there is only one-lane of traffic moving on a bridge. If a bridge is to be designed for two lanes of traffic for any type of vehicles given in the Chart, the clearance may be decided in each case depending upon the circumstances.

10. All linear dimensions are in millimetres : (1) SA. means single axle, (2) BA. means bogie axle, (3) R means revised.

APPENDIX – B

CURVES FOR LOAD DISTRIBUTION IN BRIDGE DECKS : FIG, B-1 TO B-17
(For use of these curves, reference may be made to Chapter 6)

As per Maxwell's Reciprocal Theorem, deflection at a point A due to load at B is the same as the deflection at B due to the same load at A. Similarly, the distribution coefficient, K, at a reference station, say b/2 due to load at (–) b/2 is the same as the distribution coefficient at reference station (–) b/2 due to the same load at b/2 as indicated in Fig. B-5 as (b/2, –b/2) & (–b/2, b/2). The same notation has been used in all the figures from B-1 to B-9.

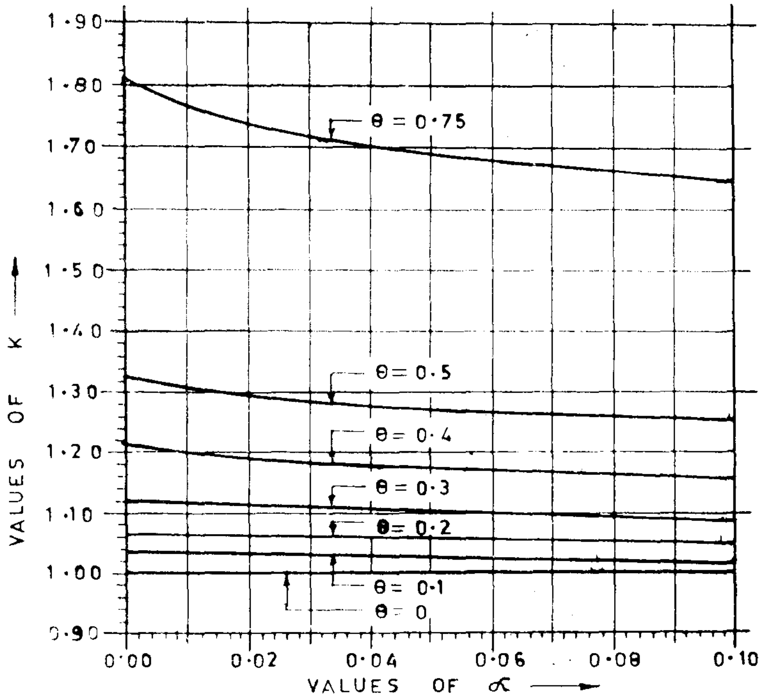


FIG. B-1: DISTRIBUTION COEFFICIENT, K AT (0,0)

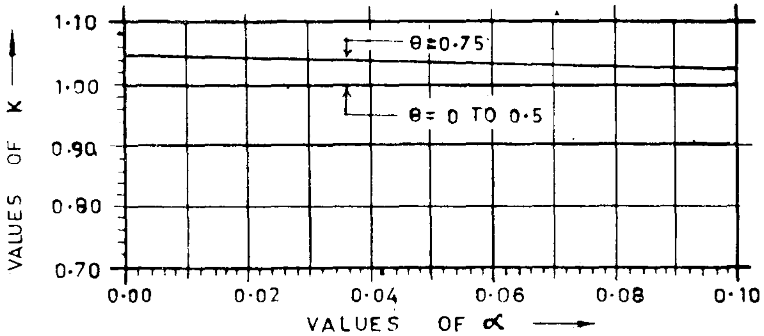


FIG. B-2: DISTRIBUTION COEFF., K AT $(0, b/2$ & $0, -b/2)$

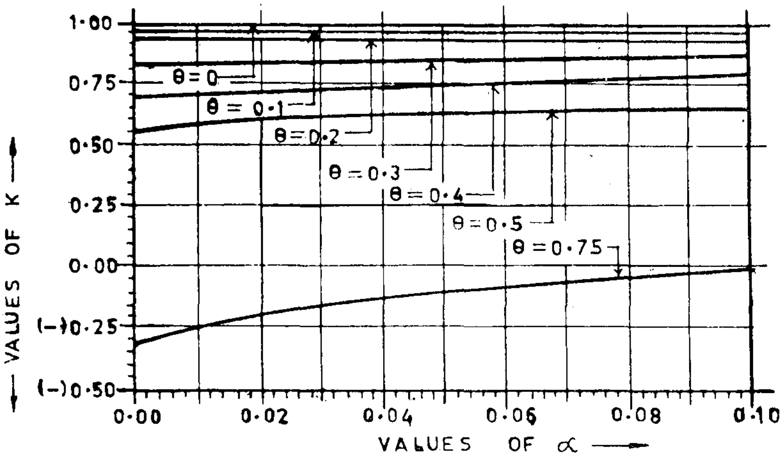


FIG. B-3 : DISTRIBUTION COEFF., K AT (0, b & 0, −b)

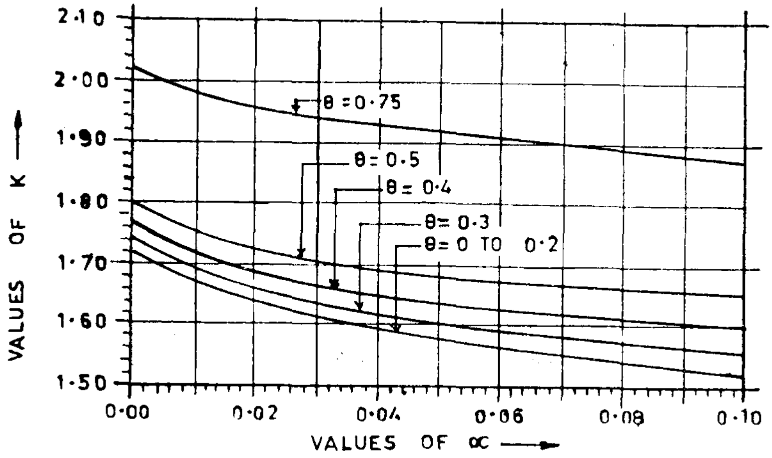


FIG. B-4 : DISTRIBUTION COEFF., K AT (b/2, b/2 & −b/2, −b/2)

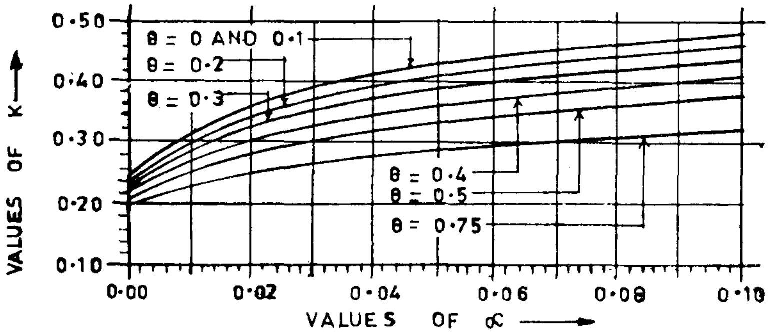


FIG. B-5 : DISTRIBUTION COEFF., K AT (b/2, − b/2 & − b/2, b/2)

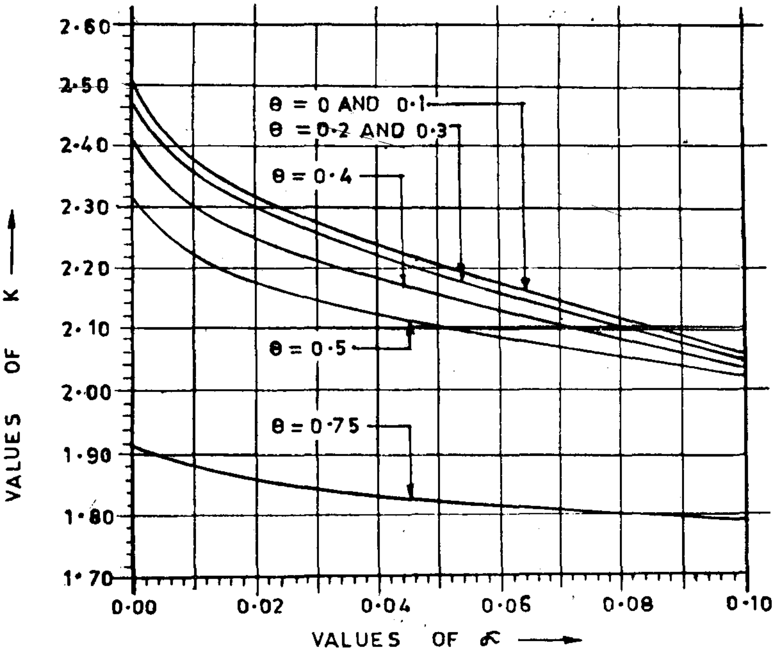


FIG. B-6: DISTRIBUTION COEFF., K AT (b/2, b & −b/2, −b)

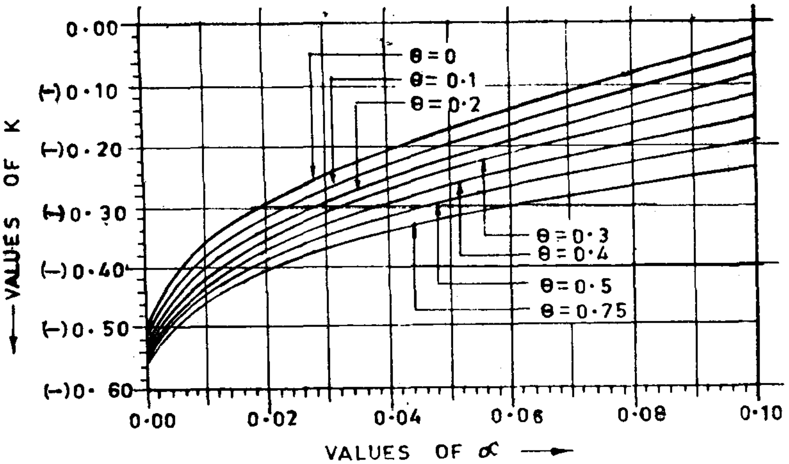


FIG. B 7: DISTRIBUTION COEFF., K AT (b/2, −b & −b/2, b)

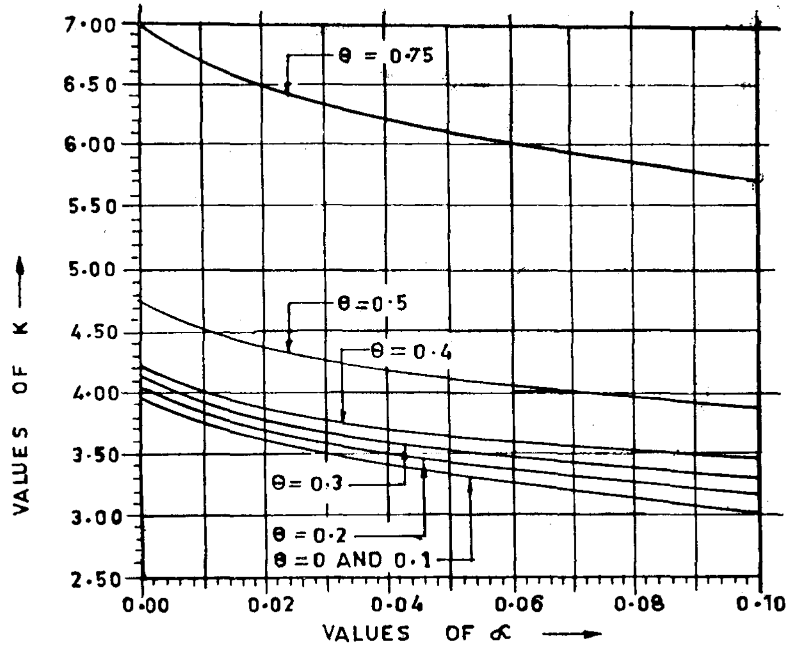


FIG. B-8: DISTRBUTION COEFF., K AT (b, b & b, b)

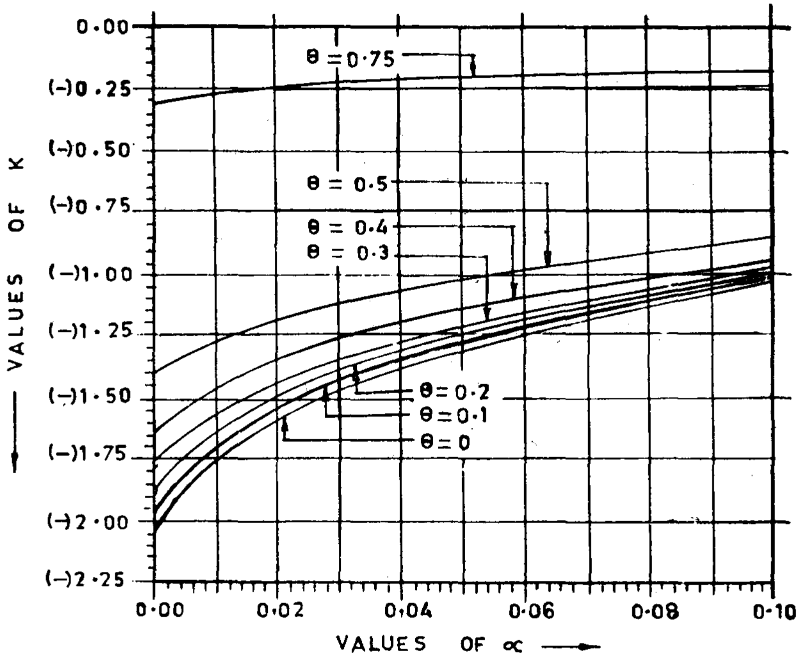


FIG. B-9: DISTRIBUTION COEFF. K AT (b, –b & –b, b)

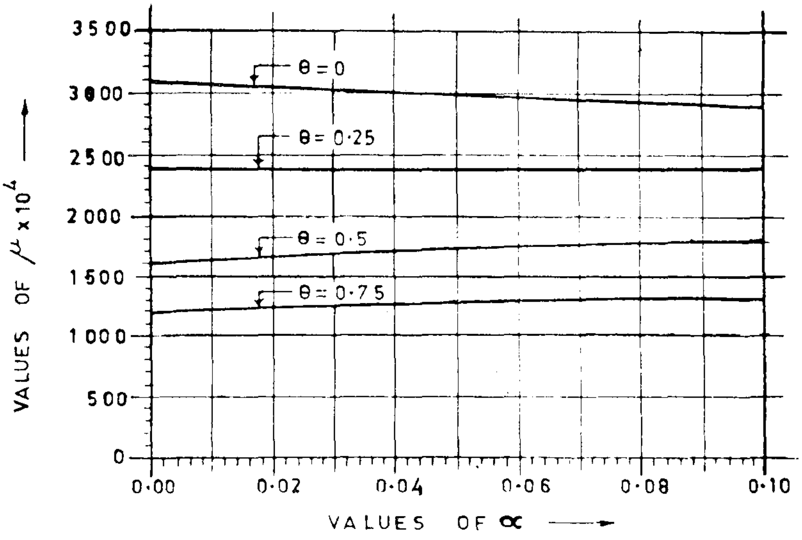


FIG. B-10: TRANSVERSE MOMENT COEFF., AT (0,0)

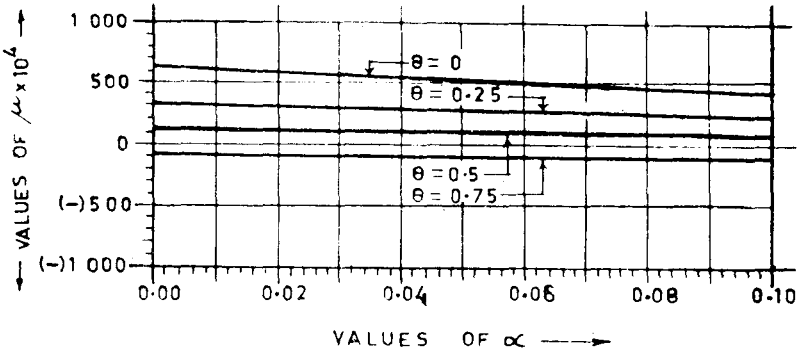


FIG. B-11: TRANSVERSE MOMENT COEFFICIENT, μ AT $(0, b/2)$ & $(0, -b/2)$

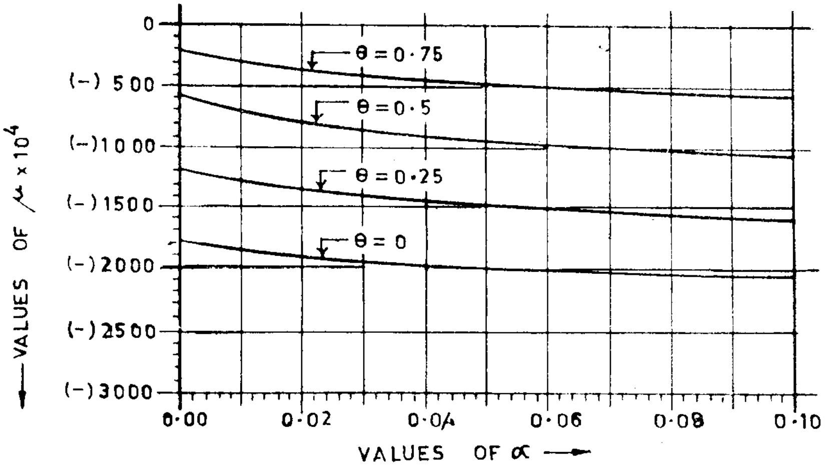


FIG. B-12: TRANSVERSE MOMENT COEFF., μ AT (0, b) & (0, −b

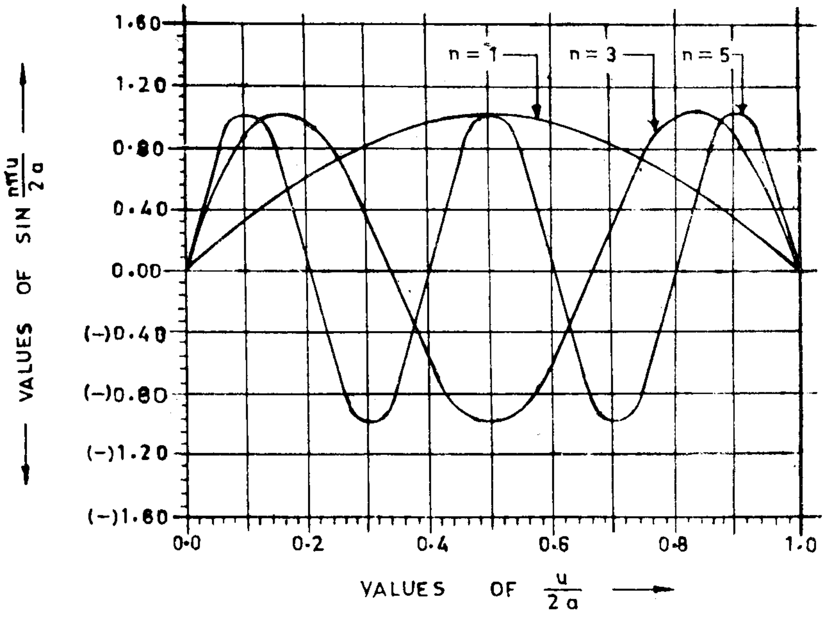


FIG. B-13: CURVES FOR VALUES OF SIN $\frac{n\pi u}{2a}$ FOR IRC CLASS A OR CLASS B LOADING

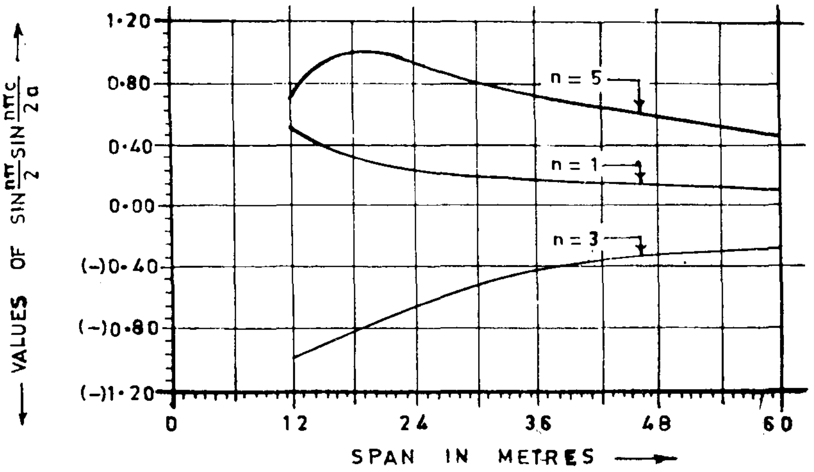


FIG. B-14: CURVES FOR VALUES OF $SIN \frac{n\pi}{2} SIN \frac{n\pi c}{2a}$ FOR IRC CLASS AA(TRACKED) LOADING

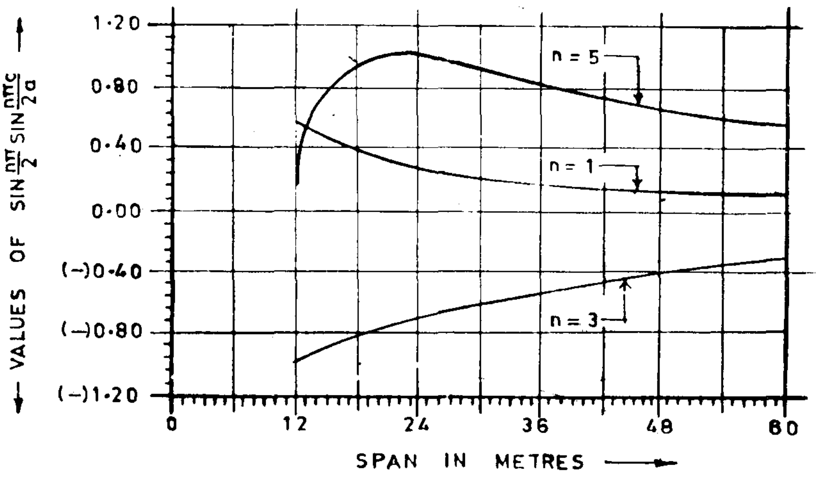
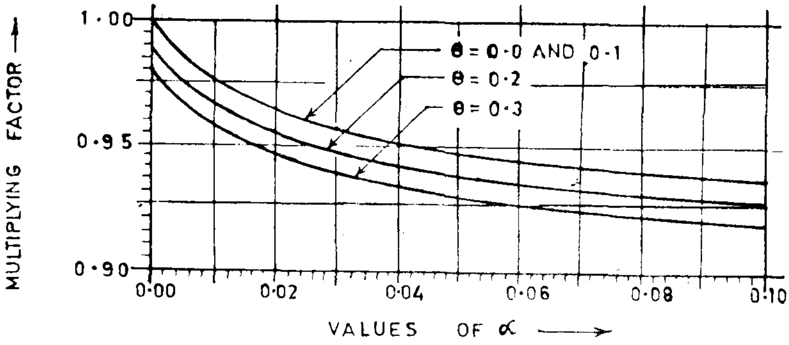
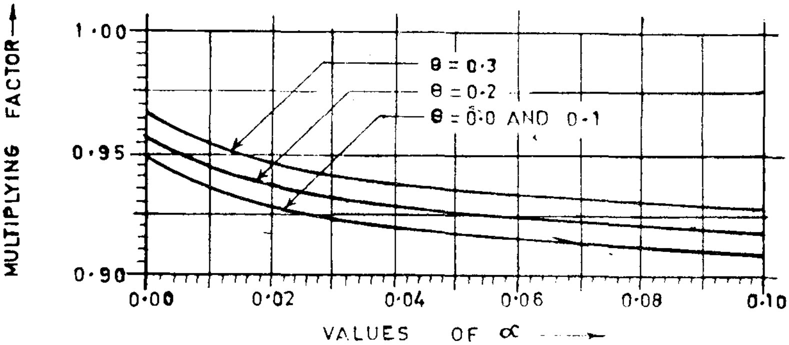


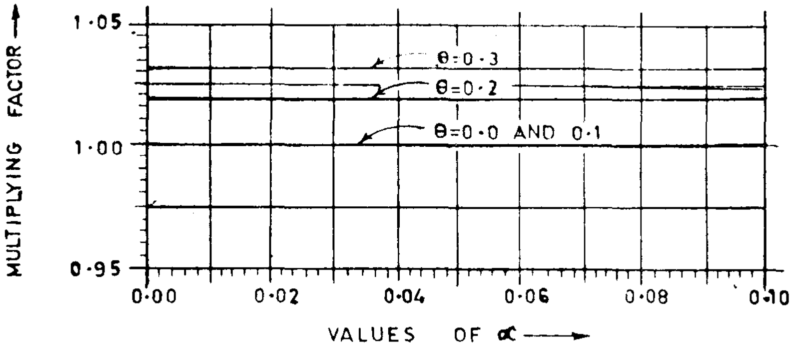
FIG. B-15: CURVES FOR VALUES OF $SIN \frac{n\pi}{2} SIN \frac{n\pi c}{2a}$ FOR IRC CLASS 70R (TRACKED) LOADING



(a) Curves for outer girder

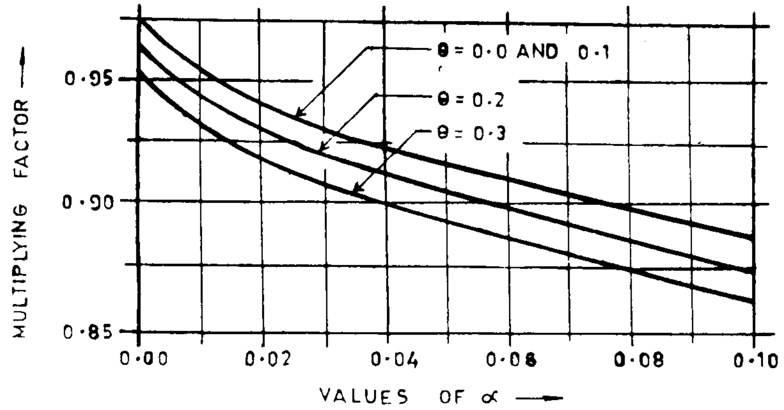


(b) Curves for inner girder

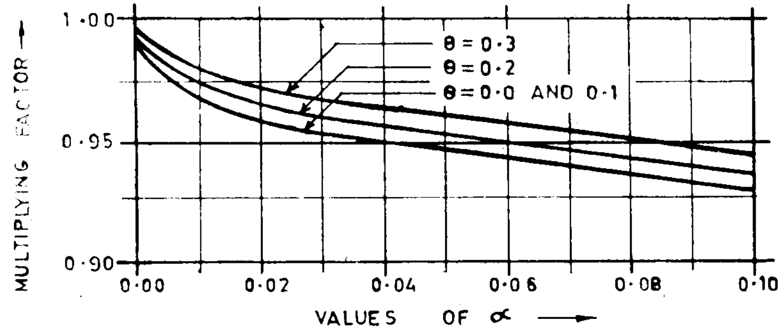


(c) Curves for central girder

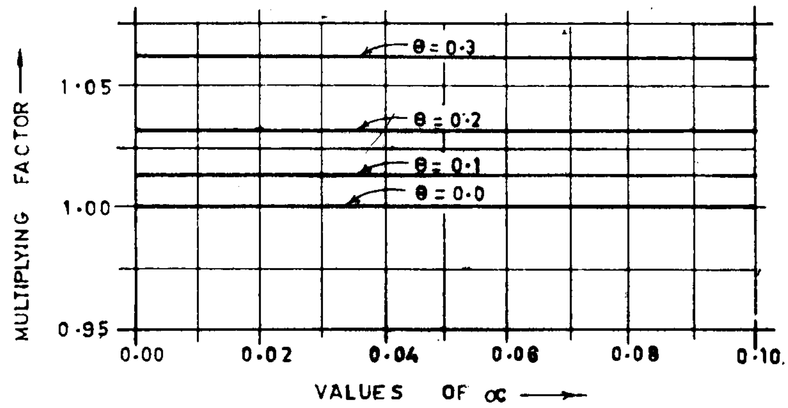
FIG. B-16: MULTIPLYING FACTORS FOR CLASS A LOADING



(a) Curves for outer girder



(b) Curves for inner girder



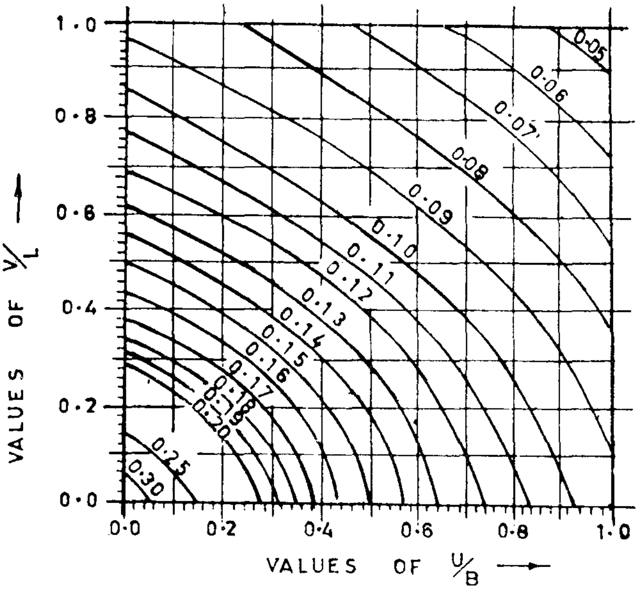
(c) Curves for central girder

FIG. B-17: MULTIPLYING FACTORS FOR CLASS AA (TRACKED) OR CLASS 70R (TRACKED) LOADING

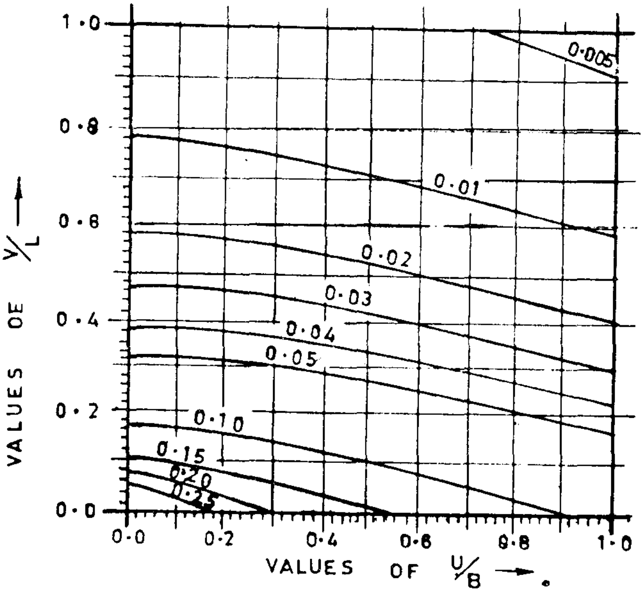
APPENDIX — C

PIGEAUD'S CURVES :- FIG. C-1 TO C-8

(The use of these curves has been illustrated in Chapter 8)

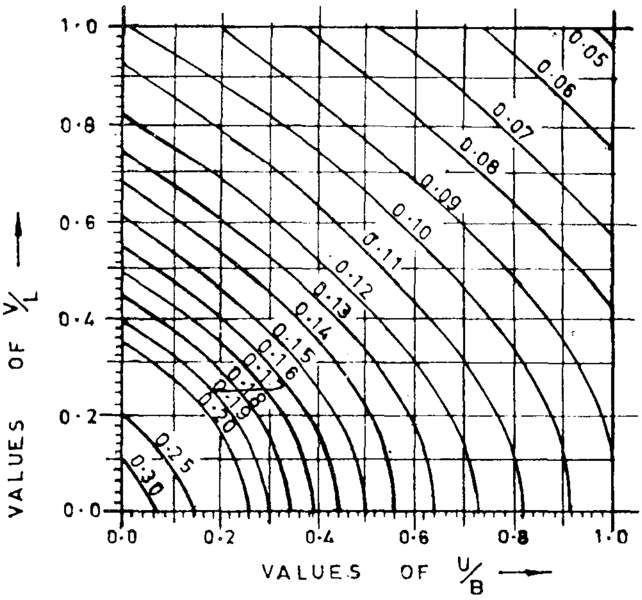


(a) Moment coefficient, m_1

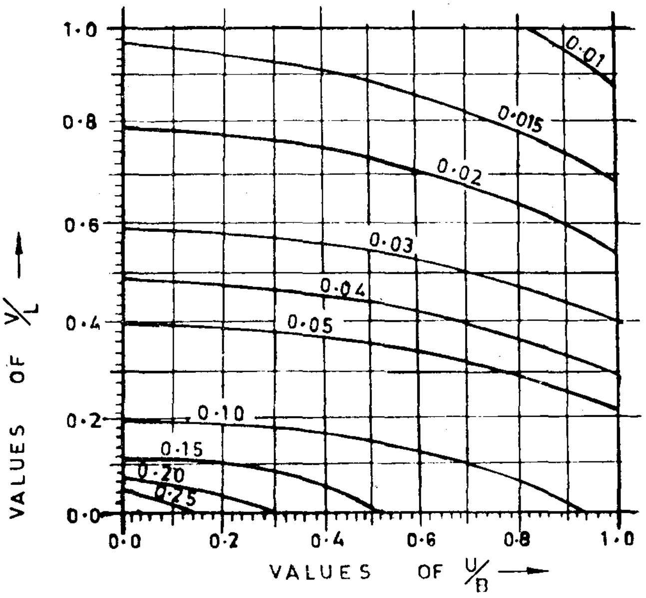


(b) Moment coefficient, m_2

FIG. C-1: MOMENT COEFF., m_1 & m_2 FOR $K = 0{\cdot}4$

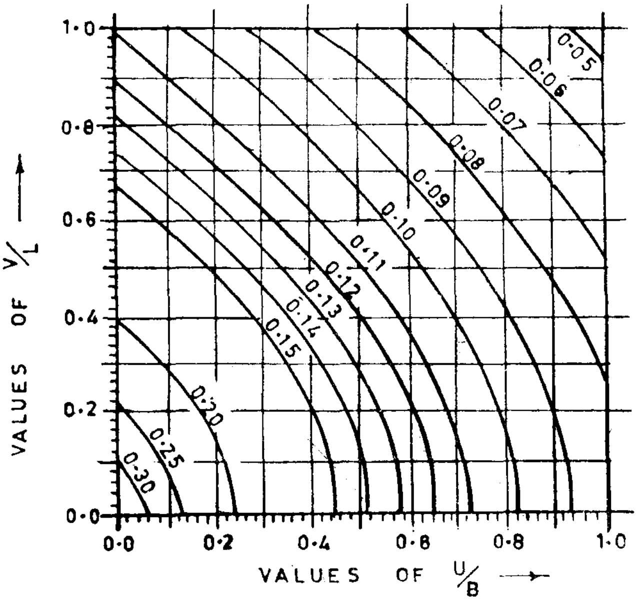


(a) Moment coefficient, m_1

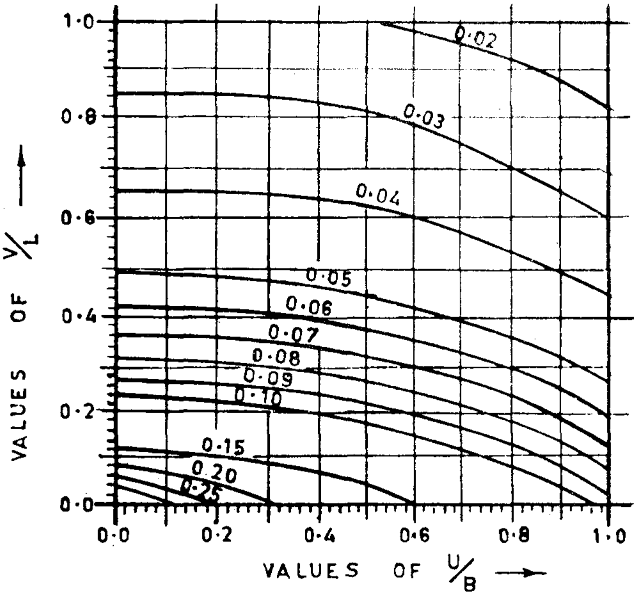


(b) Moment coefficient, m_2

FIG. C-2: MOMENT COEFF., m_1 & m_2 FOR K= 0·5

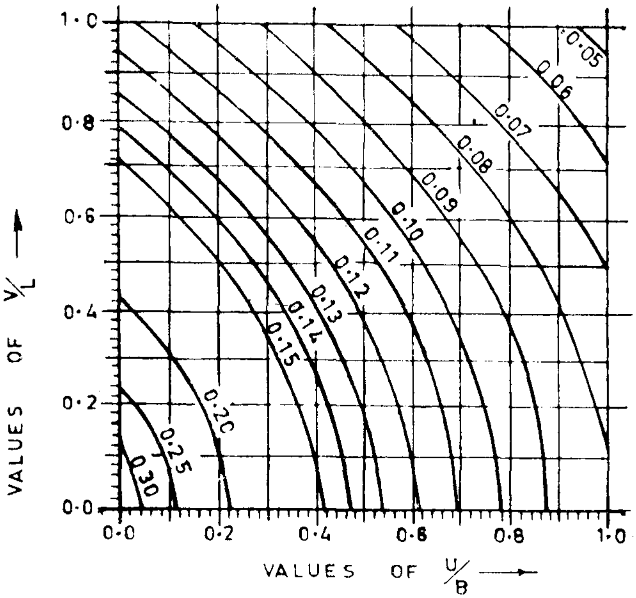


(a) Moment coefficient, m_1

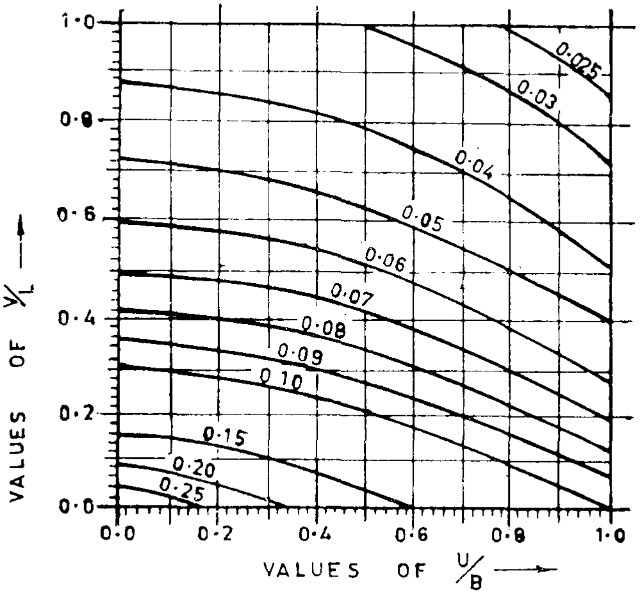


(b) Moment coefficient, m_2

FIG. C-3: MOMENT COEFF., m_1 & m_2 FOR K=0·6

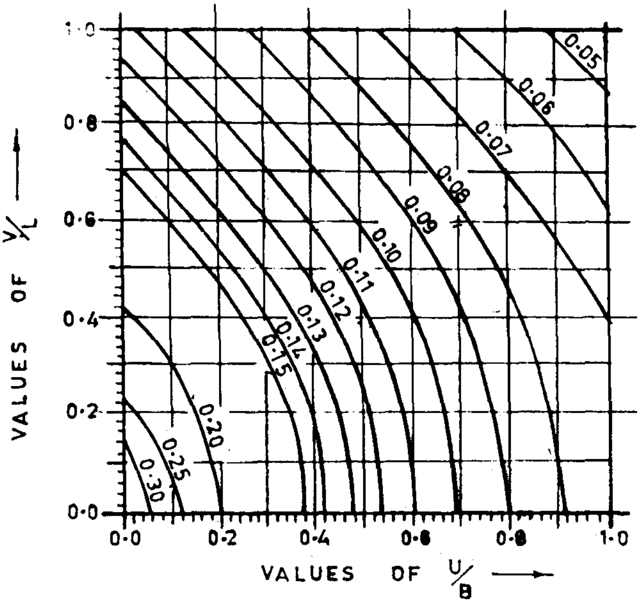


(a) Moment coefficient, m_1

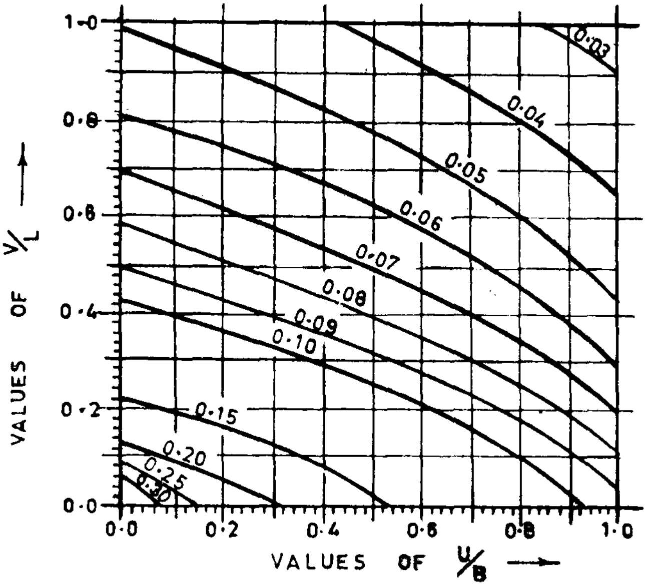


(b) Moment coefficient, m_2

FIG. C-4: MOMENT COEFF., m_1 & m_2 FOR K = 0·7

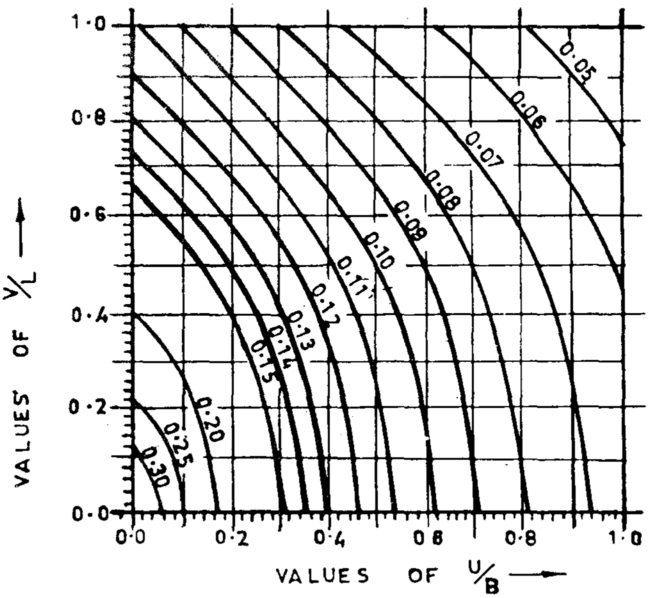


(a) Moment coefficient, m_1

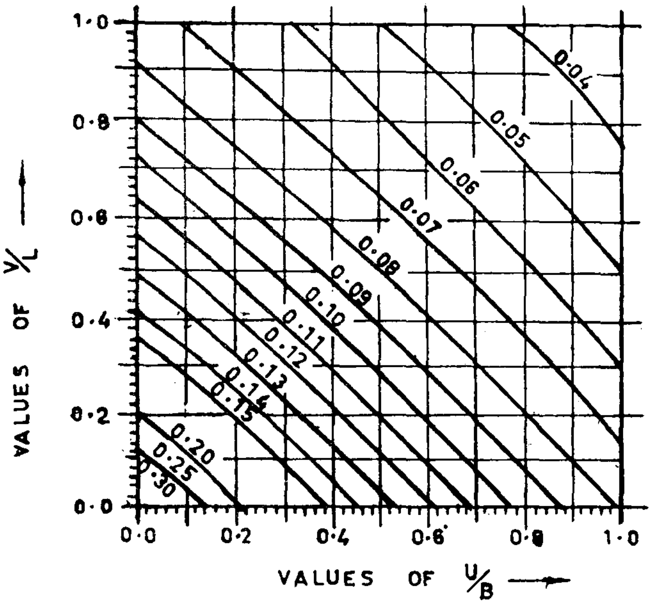


(b) Moment cofficient, m_2

FIG. C-5: MOMENT COEFF., m_1 & m_2 FOR K=0·8



(a) Moment coefficient, m_1



(b) Moment coefficient, m_2

FIG. C-8: MOMENT COEFF., m_1 & m_2 FOR K = 0·9

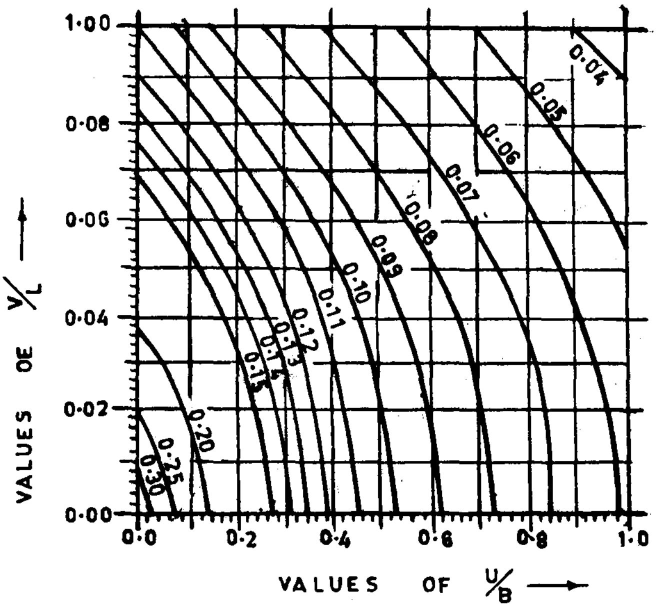


FIG. C-7: MOMENT COEFF., m_1 OR m_2 FOR K=1·0

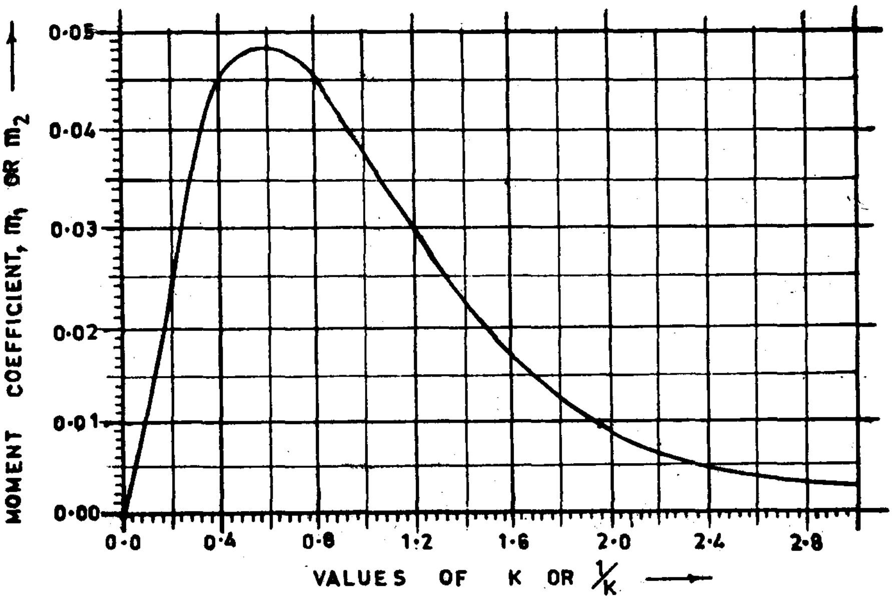


FIG. C-8: MOMENT COEFFICIENTS FOR SLABS FULLY LOADED WITH UDL: COEFF. m_1 IS FOR K AND COEFF. m_2 IS FOR $1/K$

Appendix – D

PROFORMA FOR BRIDGE INSPECTION REPORT (VIDE CH-24)

(Extract from the Indian Roads Congress Special Publication 18, "Manual for Highway Bridge Maintenance Inspection", New Delhi, 1978)

1 **GENERAL**
1.1 Name of bridge/No. of the bridge, Name of river
1.2 Name, No. of highway, bridge location

2 **TYPE OF BRIDGE**

3 **DATE OF LAST INSPECTION**

4 **APPROACHES**
4.1 Condition of pavement surface (report unevenness, settlement, cracking, pot holes, etc.)
4.2 Side slopes (report pitched or unpitched, condition of pitching/turfing, any signs of slope failure etc.)
4.3 Erosion of embankment by rain cuts or any other damage to embankment.
4.4 Approach slab (report settlement, cracks, movement, etc.)
4.5 Approach geometrics (report whether it satisfies the standards as in force).

5 **PROTECTIVE WORKS**
5.1 Type (mention whether guidebund or protection around abutments or spurs)
5.2 Report damage of the layout, cross-section profile (check whether the layout and the cross-sections are in order)
5.3 Report condition of slope pitching, apron and toe walls indicating the nature of damage if any (check for proper slope, thickness of pitching in the slopes, width and thickness of apron, erosion of toe walls, etc.)
5.4 Report condition of floor protection works, indicate nature of damage if any (condition of impervious floor, flexible apron, curtain walls, etc.)
5.5 Extent of scour (report any abnormal scour)
5.6 Reserve stone material (check against specified quantity)

6 **WATERWAY**
6.1 Report presence of obstruction, undergrowth, etc.
6.2 Report maximum observed scour and locationa and compare with the design values.

6.3 Report any abnormal change in flow pattern.
6.4 Report maximum flood level observed during the year and mark the same on the pier/abutment both on the U/S and D/S.
6.5 Report abnormal afflux if any.
6.6 Report adequacy of waterway.

7 **FOUNDATIONS**
7.1 Report settlement, if any.
7.2 Report cracking, disintegration, decay, erosion, cavitation, etc.
7.3 Report damage due to impact of floating bodies, boulders, etc.
7.4 For subways, report seepage if any, damage to the foundations, etc.

8 **SUBSTRUCTURE (piers, abutments and wingwalls)**
8.1 Report efficiency of drainage of the backfill behind abutments (check functioning of weep holes, evidence of moisture on abutment faces, etc.).
8.2 Report cracking, disintegration, etc.
8.3 For subways, report condition of side retaining walls like cracking, disintegration, etc. and seepage, if any.

9 **BEARINGS**
9.1 **Metallic Bearings**
9.1.1 Report general condition (check rusting, cleanliness, seizing of plates).
9.1.2 Functioning (report excessive movement, tilting, jumping off guides).
9.1.3 Greasing/oil bath (report date of last greasing/oil bath and whether to be redone or not).
9.1.4 Report cracks in supporting member (abutment cap, pier cap, pedestal).
9.1.5 Report effectiveness of anchor bolts (check whether they are in position and tight).
9.2 **Elastomeric Bearings**
9.2.1 Report condition of pads (oxidation, creep, flattening, bulging, splitting).
9.2.2 Report general cleanliness.
9.3 **Concrete Bearings**
9.3.1 Report any signs of distress (cracking, spalling, disintegrating).
9.3.2 Report any excessive tilting.

10 **SUPERSTRUCTURE**
10.1 **Reinforced concrete and prestressed concrete members**
10.1.1 Report spalling, distintegration or honey-combing, etc.
10.1.2 Report cracking (pattern, location, explain preferably by plotting on sketch)
10.1.3 Report corrosion of reinforcement, if any.
10.1.4 Report damages if any due to moving vehicles.
10.1.5 Report condition of articulation (cracks, if any).
10.1.6 Report perceptible vibration, if any.
10.1.7 Report excessive deflections or loss of camber, if any (measure at same point each time).
10.1.8 Report cracks in end anchorage zone (for prestressed concrete members).
10.1.9 Report deflection at central hinge, tip of cantilever for cantilever bridges.

10.2 **Steel Members**
10.2.1 Report condition of paint.
10.2.2 Report corrosion, if any.
10.2.3 Report perceptible vibrations, if any.
10.2.4 Report on alignment of members.
10.2.5 Report condition of connection (adequacy of rivets, bolts or worn out welds, report specially on connection of stringers to cross-girders, cross girders to main girders, gussets or splices, etc.)
10.2.6 Report camber and deflection.
10.2.7 Report buckling, if any.
10.2.8 Report on the cleanliness of members and joints (check choking of drainage holes provided in the bottom booms).
10.3 **Masonry Arches**
10.3.1 Report condition of joints, mortar, pointing, masonry etc.
10.3.2 Profile report flattening by observing rise of the arch at centre and quarter points.
10.3.3 Report cracks if any (indicate location, pattern, extent, depth, explain by sketches).
10.3.4 Check drainage of spandrel fillings (report bulging of spandrel walls, if any).
10.3.5 Check growth of vegetation.
10.4 **Timber Members**
10.4.1 Report condition of paint.
10.4.2 Check decay, wear and tear, structural defects needing immediate replacement, if any.
10.4.3 Report condition of joints, splices, spikes etc.
10.4.4 Report excessive sag, if any.
10.5 **Suspension Bridges**
10.5.1 Report condition of cables.
10.5.2 Report condition of suspenders and their connectors.
10.5.3 Report condition of structural steel.
10.5.4 Report condition of painting.
10.5.5 Report excessive oscillations, if any, requiring need of guy ropes.
10.5.6 Report looseness of joints, bolts, rivets, welds.
10.5.7 Report condition of anchors, evidence of movement.
10.5.8 Report condition of towers and saddles (verticality, lateral support).

11 **EXPANSION JOINTS**
11.1 Functioning (report cracks in deck in the existing gap and approximate temperature).
11.2 Report condition of sealing material (for neoprene sealing material, check for splitting, oxidation, creep, flattening, bulging and for bitumen filler, check for hardening, cracking, etc.).
11.3 Report secureness of the joints.
11.4 Top sliding plate (report corrosion, damage to welds, etc.).
11.5 Locking of joints (report locking of joints especially for finger type expansion joints).

11.6 Check for debris in open joints.
11.7 Report rattling, if any.

12 **WEARING COAT (concrete/bitumen)**
12.1 Report surface condition (cracks, spalling, disintegration, pot holes etc.).
12.2 Report evidence of wear (tell tale rings, check for thickness as against actual thickness, report date of last inspection).

13 **DRAINAGE SPOUTS**
13.1 Check clogging, deterioration and damages, if any.
13.2 Check the projection of the spout on the underside (see whether structural members are being affected).
13.3 Report adequacy thereof.
13.4 For subways, report about adequacy of pumping arrangements etc.

14 **HANDRAILS**
14.1 Report general condition, check expansion gaps, missing parts, if any, etc.
14.2 Report damage due to collision.
14.3 Check alignment (report any abruptness in profile).

15 **FOOTPATHS**
15.1 Report general condition (damage due to mounting of vehicles).
15.2 Report missing footpath slabs.

16 **UTILITIES**
16.1 Report leakage of water and sewage pipes
16.2 Report any damage to telephone and electric cables.
16.3 Report condition of lighting facilities.
16.4 Report damages due to any other utilities.

17 **BRIDGE NUMBER**
17.1 Report condition of painting.

18 **AESTHETICS**
18.1 Report any visual intrusion (bill boards, paints on structural members, etc.).

19 Report whether maintenance recommended during last inspection has been done or not (give details).

20 Maintenance and improvement recommendations

Sl. No.	Item needing attention	Action recommended	Time when to be completed	Remarks

21 **Certificate to be accorded by the inspecting official**
Certified that I have personally inspected this bridge.

Signature

Date :

Designation of the Inspecting Officer.

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