LONG SPAN PRESTRESSED CONCRETE BRIDGES
OF SEGMENTAL CONSTRUCTION
STATE OF THE ART

by

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Bridges of Segmental Construction

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P R E F A C E

This report is the first in a series which will summarize a detailed investigation of the various problems associated with formulation of design procedures for long-span prestressed concrete bridges of segmental construction. This report presents a critical summary of the present "state of the art" for such structures. In preparation of this report, all available published engineering articles on the subject were reviewed, and a number of interviews were held with people actively connected with several European projects. An attempt has been made to summarize and synthesize the main items which appear to govern practical design and construction procedures without a lengthy repetition of the available literature. A selected bibliography of important articles in the field is included.

This work is a part of Research Contract 3-5-69-121 entitled "Design Procedures for Long-Span Prestressed Concrete Bridges of Segmental Construction." The studies described herein were conducted as a part of the overall research program at The University of Texas Center for Highway Research. The work was sponsored jointly by the Texas Highway Department and the Federal Highway Administration under an agreement between The University of Texas at Austin and the Texas Highway Department.

Liaison with the Texas Highway Department was maintained through the contact representatives, Mr. Robert L. Reed and Mr. Wayne Henneberger; Mr. D. E. Harley was the contact representative for the Federal Highway Administration.

This study was directed by Dr. John E. Breen, Professor of Civil Engineering, and Dr. Ned H. Burns, Associate Professor of Civil Engineering. The "state of the art" summary phase was the overall responsibility of G. C. Lacey, Research Engineer, Center for Highway Research. Important contributions to the chapter on "Methods of Analysis" were made by R. C. Brown, Research Engineer, Center for Highway Research.

ABSTRACT

A number of examples of long-span prestressed concrete bridges, constructed using segmental precast box girder elements, are reviewed. Cross-sectional shapes, prestressing system patterns, and main techniques of precasting, erection, and jointing are examined in detail. A selected bibliography of major references is included.

S U M M A R Y

This report examines in detail the application of segmental precast box girder construction in achieving long spans in bridge structures. Numerous examples from throughout the world indicate that such construction can provide an effective means of achieving long spans in the 100 to 500 ft. range. Using segmental precasting the bridge structure is manufactured in a number of short units which during erection are joined together, end to end, and post-tensioned to form the completed superstructure. Efficient casting techniques have been developed to obtain the tolerances required both for concrete jointing and for epoxy resin jointing. In the latter case the required precision is obtained by casting the segments one against the other.

The most frequent methods of construction are erection on falsework and cantilever erection. The former is the simpler method. The temporary supports can be set at close intervals without causing disruption of the space utilization beneath the structure. Otherwise, cantilever construction or a combination of both methods may be more suitable.

Detailed information is given of the types of prestressing systems, jointing techniques, and erection methods that have found successful application in the long span bridges of this type throughout the world. It is concluded that this is a promising method which can find application in American bridge practice.

I M P L E M E N T A T I O N

This report presents a critical summary of the present state-of-the-art for design and construction procedures for long span, prestressed concrete bridges of segmental construction. The survey of the application of this type of construction throughout the world indicates that there is a very strong possibility that this construction procedure can provide an effective means of achieving long spans.

Experience in Europe, Asia, and Australia indicates that no insurmountable technological problems exist for either the design or construction of segmentally precast box girder structures. However, it will be necessary to refine design criteria, design procedures, and construction procedures to suit American safety, industrial, and economic conditions. There is a need for further study of the relative advantages in economy for both box girder and possible alternate structural systems. However, it appears that the great advantage of precast box girders is that they can be used for a very large range of spans, with all of the advantages of industrialization and quality control that are found in the precast industry. It looks as if this type of bridge structure could be a major competitive force in the 100 to 500 ft. span range, with the additional advantage that field concreting is almost eliminated.

An additional and significant benefit of this type of construction is that it is possible to achieve very high speeds of erection of long span structures and to do so with relatively little disruption of water or highway traffic under the structure being erected. Applications are reported where this procedure was utilized to erect bridges above busy expressways with minimal interruption in their traffic flow. This could be of significant importance in providing better service and increased safety in areas where major improvements are having to be made in existing expressway systems.

In order to introduce this construction system into Texas Highway Department practice, it will be necessary to develop an applicable design criteria, design procedures, and construction procedures which will suit local contractor capabilities and material availabilities. The design criteria and procedures should be checked by careful observation of a physical model and a prototype structure selected to provide a pilot study for both field application and economic information. The state of knowledge is such that rapid development of this type of construction seems practical.

CONTENTS

Part Page

LIST OF TABLES AND FIGURES

CHAPTER 1

INTRODUCTION

In highway bridge construction there is an increasing trend toward the use of longer spans. This trend is the result of a number of different requirements relating to safety, economy, function, and aesthetics.

The February 1967 report of the special AASHO Traffic Safety Committee calls for the "adoption and use of two-span bridges for overpasses crossing divided highways . . . to eliminate the bridge piers normally placed adjacent to the outside shoulders."[38,48*] To comply with these safety criteria, two-span, three-span, and four-span overpasses will be required with spans up to 180 ft.

In the case of elevated urban expressways, long spans will facilitate access and minimize obstruction. With stream crossings, even longer spans in the 300 ft. range will sometimes be necessary. When all factors are considered the trend toward long span bridges seem irreversible.

Precast I-girders, of the type now widely used for spans up to the 120-ft. range, will not be adequate for these longer spans. It was proposed in Ref. 48 that AASHO Type VI I-girders might be used for simple spans up to 140 ft. and continuous spans up to 160 ft. (where precast lengths of 80 ft. are joined together by splicing). To obtain greater spans than this the use of haunched sections or inclined piers was also proposed.

A substantial number of long span bridges have been constructed throughout the world utilizing prestressed concrete box girders. This form results in a very compact structural member, which combines high flexural strength with high torsional strength. The box girders may be either

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*Superscript numbers indicate references listed in the Bibliography at the end of this report.

cast-in-place or precast in segments. Their suitability for even very long spans can be seen from the Bendorf Bridge in West Germany, which was cast-in-place and has a span of 682 ft. In the United States, cast-in-place box girder bridges are being widely used by the California Division of Highways, as well as several other states.

When construction of large numbers of bridges is envisaged, as in a highway department, precasting has a number of advantages over cast-in-place construction, e.g.:

(1) Mass production of standardized girder units is possible. This is done at present with precast I-girders for shorter spans.

(2) High quality control can be attained through plant production and inspection.

(3) Greater economy of production is possible by precasting the girder units at a plant site rather than casting in place.

(4) The speed of erection can be much greater. This is very important when construction interferes with existing traffic and is most critical in an urban environment.

The objective of this report is to examine in detail one means of achieving long spans in bridge structures, namely, segmental precast box girder construction. Segmental precasting involves the manufacture of the bridge structure in a number of short units. During erection these are joined together, end to end, and post-tensioned to form the completed superstructure. The segmental pattern for a typical bridge is shown in Fig. 1. The reason for casting in short segments is essentially that box girders, unlike I-girders which have narrow width, cannot be readily transported in long sections. In addition, the short units are suited to fairly simple methods of assembly and post-tensioning. The length and weight of the segments are chosen so as to be most suitable for transportation and erection.

In the following chapters examples of long span bridges, made from segmental, precast box girder elements, are reviewed. Cross-sectional shapes, prestressing system patterns, and the techniques of precasting, erection, and jointing are examined in detail.

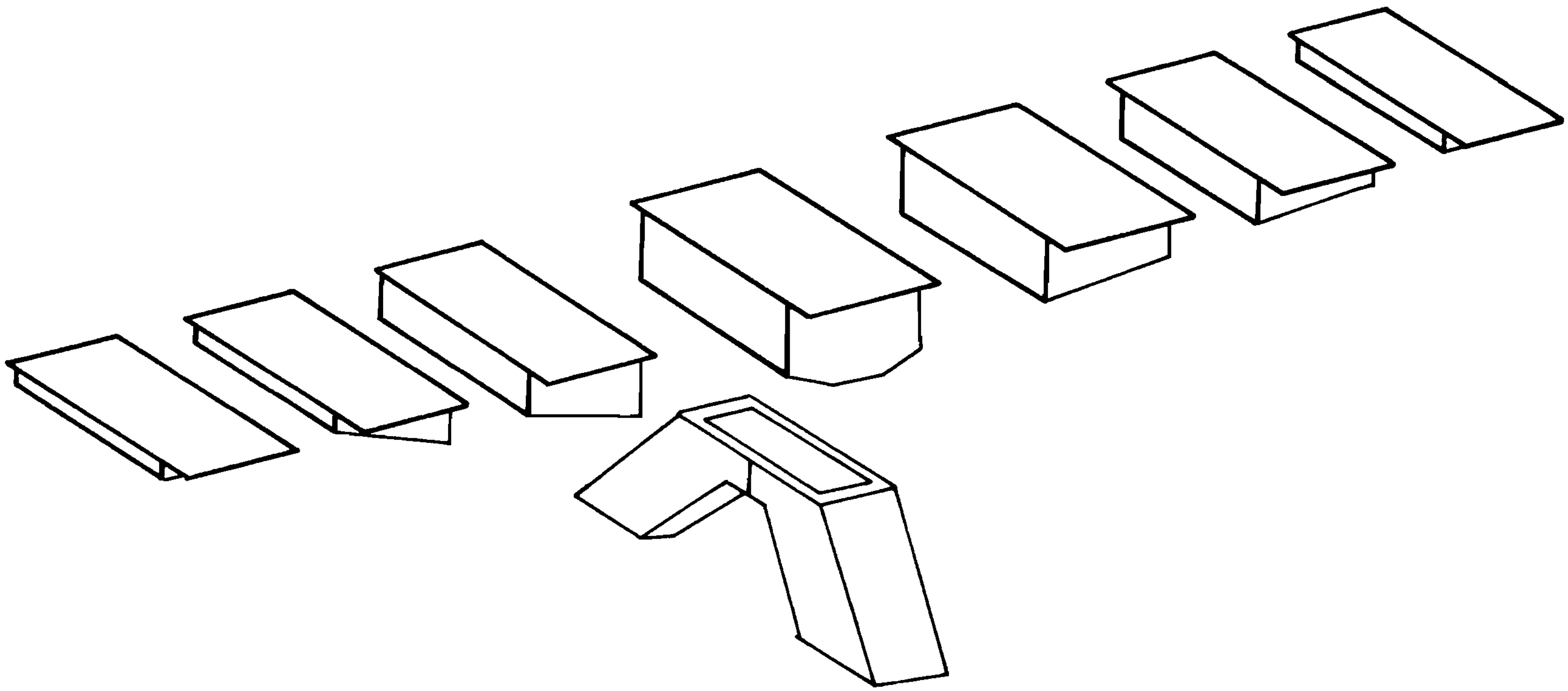


Fig. 1. Superstructure of the Oosterschelde Bridge, The Netherlands.

CHAPTER 2

TYPES OF CROSS SECTION AND PRESTRESSING SYSTEMS

Dimensions and details of a number of prestressed box girder bridges, constructed from precast segments, are summarized in Table 1* and the cross sections of some are shown in Figs. 1 to 15.

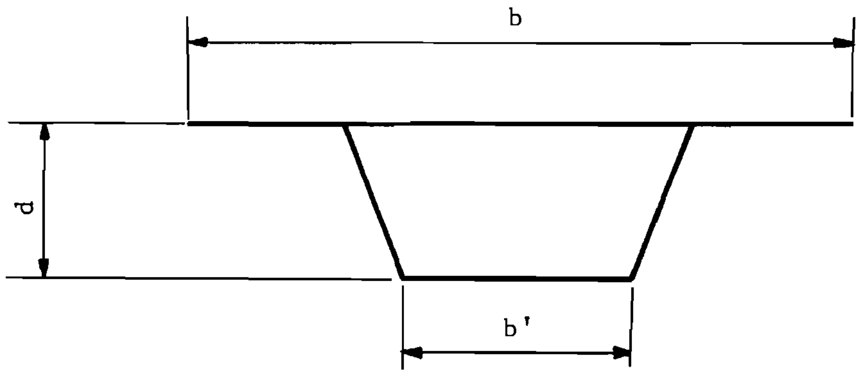
With the exception of the Hammersmith Flyover the superstructures of these bridges generally conform to three main types: (a) single cell box girder, (b) pair of single cell box girders connected by the deck slab, and (c) multicell box girder. These types are sketched in Fig. 2. The prestressing systems used depend partly on the type of cross section, the structural system, and the method of construction, but vary greatly from bridge to bridge. Several of the bridges included in Table 1 are described in detail in the following sections.

2.1 Hammersmith Flyover

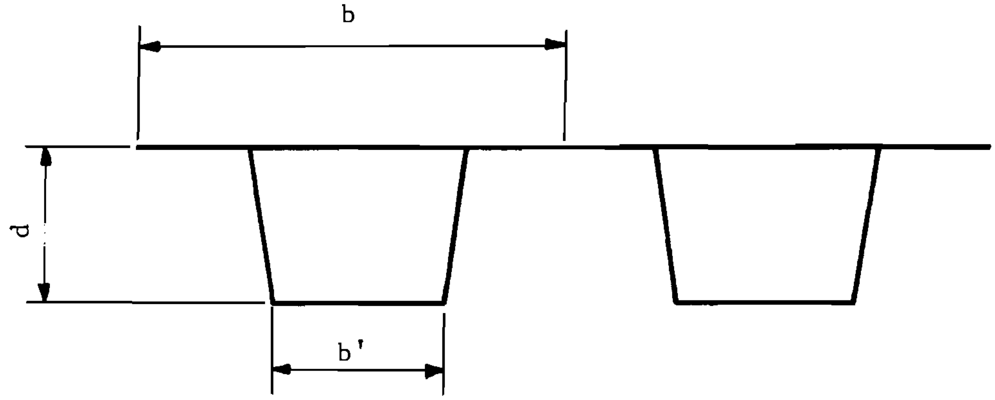
The Hammersmith Flyover in London[1,2,6] is of unique construction (see Fig. 3). The main girder element is a three-cell hollow box, 26 ft. wide, cast in segments 8 ft.-6 in. long. These alternate with 1-ft. thick precast cantilever "coathanger" sections 60 ft. wide. The sections are joined by 3 in. of cast-in-place concrete. Precast concrete slabs form the deck of the structure. The cantilever sections act as diaphragms for the box girder and also support the outer deck slab units. The girders are rigidly connected to their supporting columns, which in turn rest on roller bearings.

The Gifford-Udall system is used for longitudinal prestressing. Stranded cables, 1-1/8 in. in diameter, are arranged in four clusters of 16 each, one cluster on either side of each inner web of the girder. At

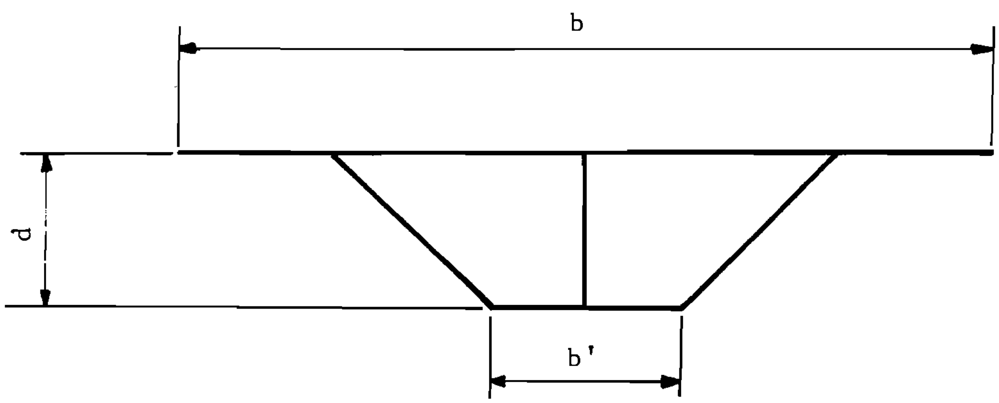
*See Appendix.



(a) Single cell box girder.



(b) Single cell boxes connected by upper slab.



(c) Two cell box girder.

Fig. 2. Box girder cross section types.

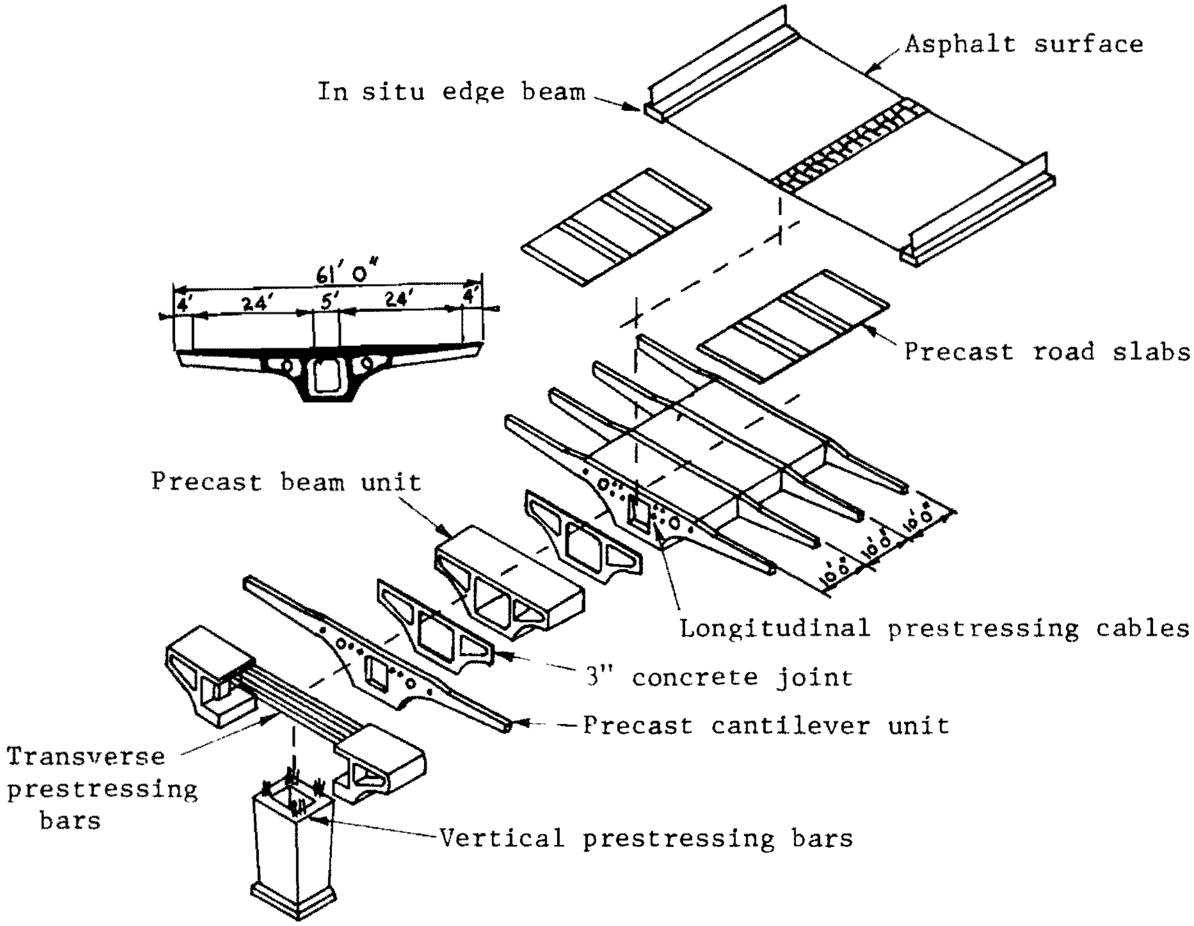


Fig. 3. Hammersmith Flyover, London.

midspan each cluster passes through a 10-in. diameter duct in the lower flange. The cables are arranged longitudinally in overlapping groups, each group passing through two successive spans and overlapping the next group by one span length (see Fig. 4). Both ends are anchored in the top flange of the box girder after passing over a column. The cables are kept in position by steel saddles over the columns and at points 25 ft. either side of midspan and the cable profile is linear between the saddles.

The beam segments that rest on the columns are prestressed transversely through the top flange and are tied to the columns with vertical prestressing.

2.2 Single Cell Box Girders

The following bridges are of this type: Ager (Austria), Margecany (Czechoslovakia), Mancunian Way (England), Oleron (France), Kakio and Konoshima (Japan), Oosterschelde (Netherlands), and Caroni (Venezuela).

The Mancunian Way, Manchester,[26,39] comprises a pair of single cell box girders, joined by a cast-in-place median strip (see Fig. 5). The upper slab of each box is cantilevered out to accommodate the roadway width. The precast segments are 7 ft.-3 in. long, with 3-in. joints of in situ concrete between. The vertical webs of the box girders are made thick enough to accommodate all the prestressing cables. There are 16 cables at each section, two layers of 4 in each web. Each span contains two sets of cables, each set being two spans long and overlapping the next by one span length. Prestressing anchorages are positioned near the quarter point of the span. The girders are supported by "Rotaflon" sliding bearings on the columns.

The Oleron Viaduct, France,[20,24,42] is a single cell box girder bridge, with the upper slab cantilevered out to a total width of 35 ft. (see Fig. 6). The precast segments are 10.8 ft. long. The girder is prestressed in the longitudinal and transverse directions. The superstructure is elastically fixed to each pier through four neoprene bearing pads.

The Oosterschelde Bridge in the Netherlands[21,31] is a single cell box girder with cantilevered upper slab (see Fig. 1). The depth tapers from 19 ft. at the piers to 7 ft. at midspan. The precast segments are

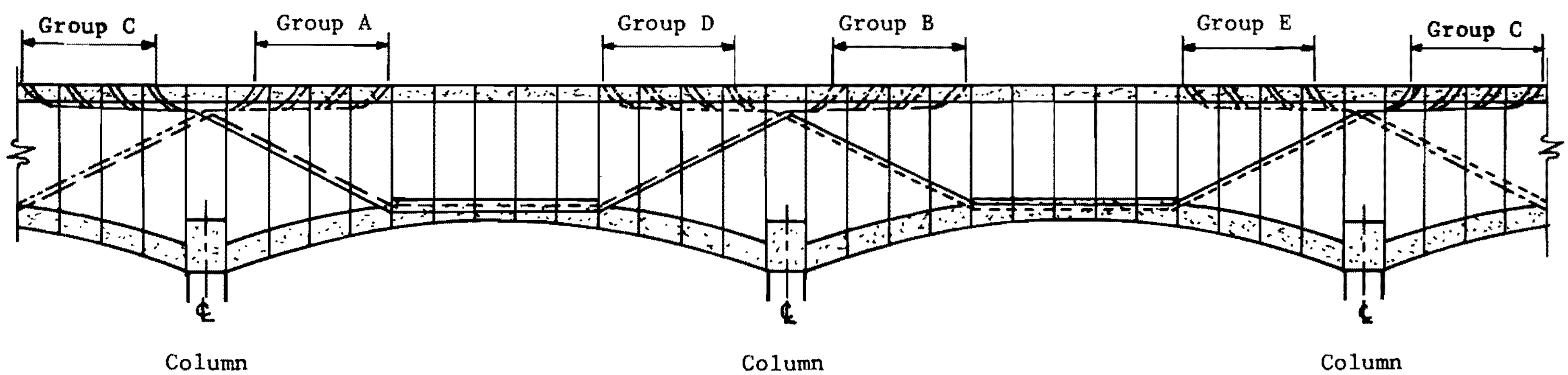


Fig. 4. Hammersmith Flyover--cable profile.

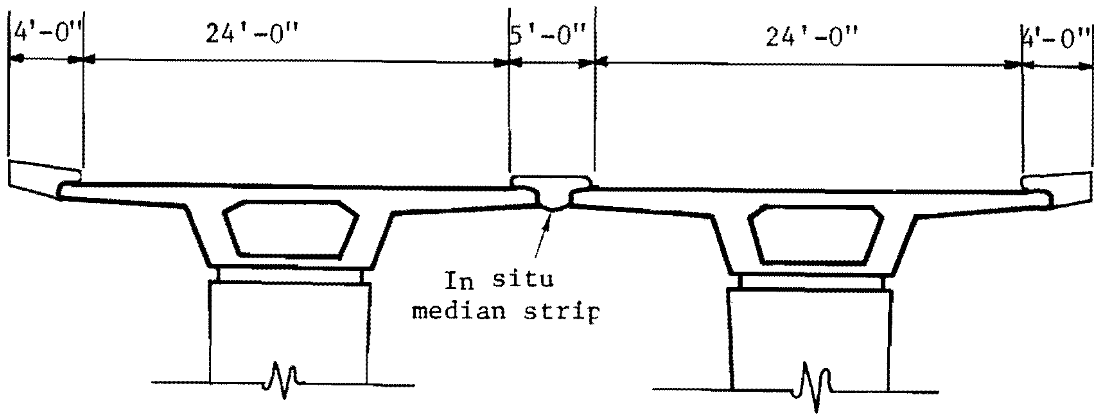


Fig. 5. Mancunian Way, Manchester

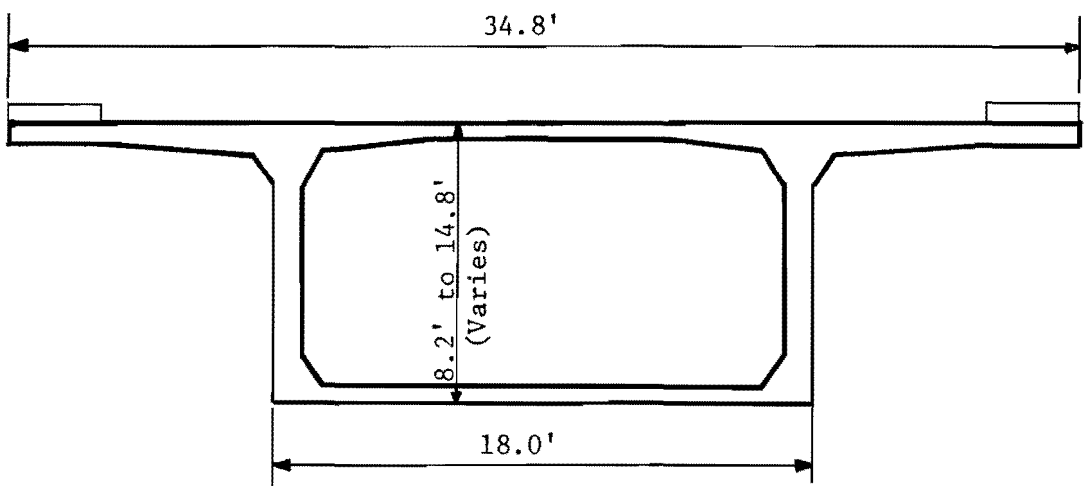


Fig. 6. Oleron Viaduct, France.

about 41 ft. long. The structure is not continuous but consists of a series of double cantilevers rigidly supported at the piers and separated by open joints at midspan. To restrict vertical movement while accommodating expansion between the cantilever ends, dowels and shock absorbers were installed at the midspan joints.

Longitudinal prestressing consists of Freyssinet cables in the deck and also a few in the lower slab near midspan. The segment walls are stressed vertically with unbonded bars and laterally with Freyssinet tendons.

The Ager Bridge in Austria[9,23] (Fig. 7) consists of a pair of independent, single cell box girders with cantilevered upper slabs and having a constant depth of 14 ft. The precast segments are 30 ft. long. The prestressing cables are external and are bonded to the web by concrete encasement and stirrups anchored in the web.

2.3 Bridges Formed from Two Single Cell Boxes Connected by the Upper Slab

This type includes the following bridges: Ondava (Czechoslovakia), Choisy-le-Roi, Pierre Benite, Courbevoie, and Pont Aval (France), Kamiosaki and Tama (Japan), and Hartelkanaal (Netherlands).

The Choisy-le-Roi Bridge, Paris,[16,33] consists of two identical half bridges (see Fig. 8). These are connected by a 4-ft. wide concrete slab with a longitudinal hinge joint on either side, and are tied together with transverse post-tensioning. The bridge has three continuous spans. The superstructure forms a portal with the two piers and is simply supported at the abutments.

Each half bridge, 44 ft. in width, consists of two box girders, 8.2 ft. in depth and cast separately in 8.2-ft. long segments. The top slab of each box is cantilevered out in both directions. After erection and longitudinal post-tensioning, the two slabs are connected rigidly with a concrete joint and transverse post-tensioning. At the piers the two box girders are braced together with a transverse box, the vertical faces of which also function as girder diaphragms, one over each wall of the V-shaped pier. There are single diaphragms at the abutments, both internally

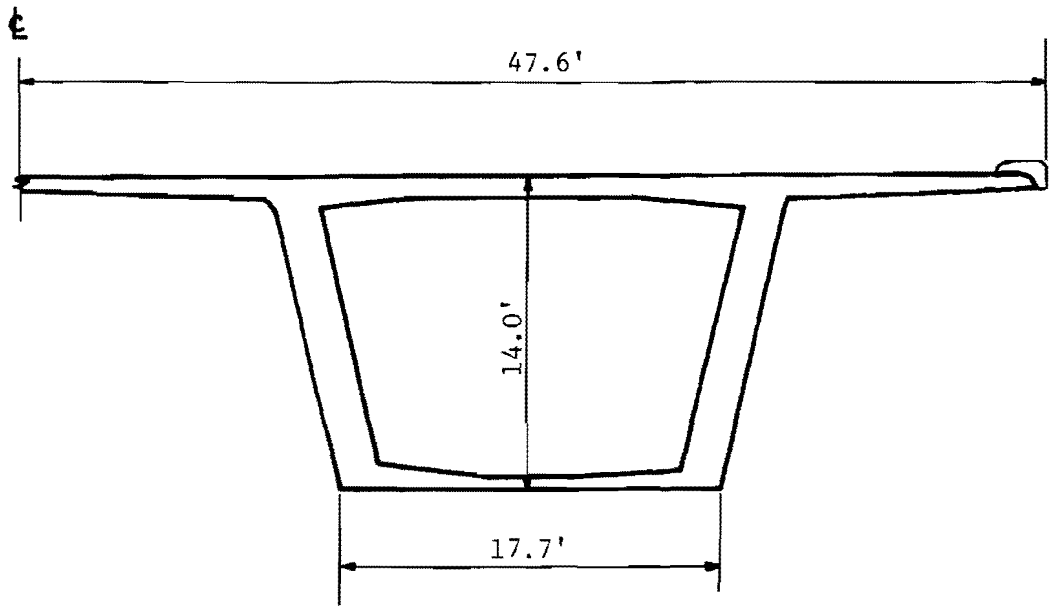


Fig. 7. Ager Bridge, Austria.

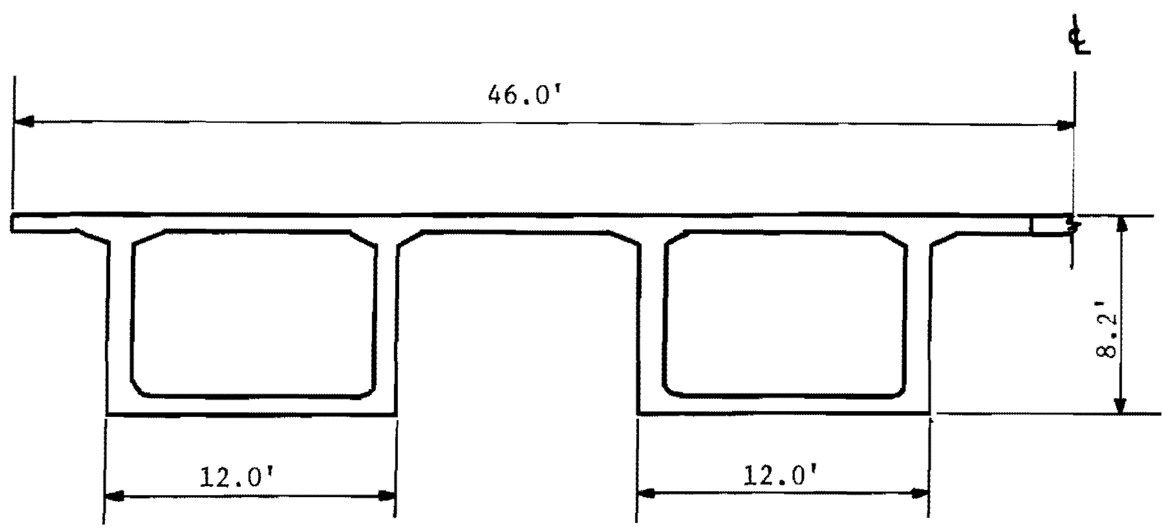


Fig. 8. Choisy-le-Roi Bridge, Paris.

and externally (i.e., between the two boxes). There are no intermediate diaphragms.

There are two sets of longitudinal prestressing cables (see Fig. 14). The primary function of one set is to withstand the negative moments during construction (by the cantilever method). Some of these cables, of varying length, run horizontally in the deck slab; the remainder have draped profiles with anchorage in the girder web. The other set of cables is located in the lower slab for positive moment near midspan. Most of these are draped and anchored in the deck slab.

The bridge on the Rhone at Pierre Benite[34] (Fig. 9) consists of a pair of single cell box girders with projecting upper slabs, rigidly connected after erection. The depth is 11.8 ft., except near the supports where it increases to approximately 14 ft. Each box is cast in segments 9.8 ft. long. The superstructure is continuous over the intermediate supporting caissons, to which it is rigidly connected, and it rests on neoprene pads at the abutments. The bridge is skewed and there is a slight curvature.

The diaphragms at the caissons are extended to form a transverse box section bracing the two box girders. There are single internal and external diaphragms at the abutments but no intermediate diaphragms. The girder segments (and the diaphragms) at the caissons and abutments are cast in place, because of the skew, whereas all other segments are prefabricated.

As in the case of Choisy-le-Roi there are two sets of longitudinal prestressing cables, for negative and positive moments, respectively. After erection and longitudinal post-tensioning, the two boxes are connected by a 3.6-ft. wide slab and the transverse post-tensioning is applied.

The superstructure of the Pont Aval, Paris,[41,52] is similar to that of Pierre Benite (see Fig.10). It consists of a pair of single cell box girders, with upper slabs rigidly connected by an 8-in. cast-in-place joint and tied together with transverse post-tensioning. The overall width is 52 ft. The depth of the girder is 11.1 ft. over the central portion, but increases to 18 ft. at the supports. The precast segments are 12.5 ft. long.

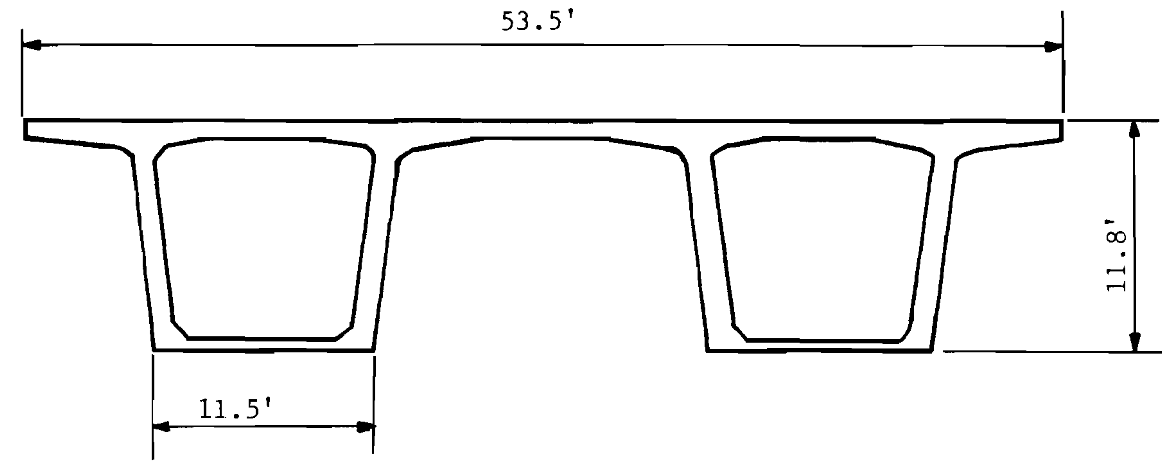


Fig. 9. Bridge on the Rhone at Pierre Benite, France.

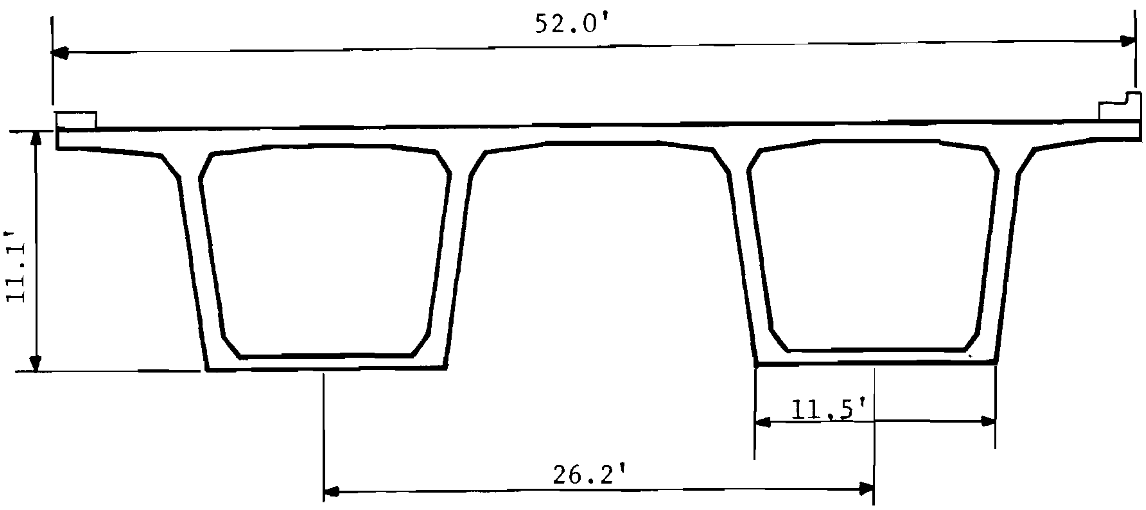


Fig. 10. Pont Aval, Paris.

The bridge has four continuous spans and is supported on neoprene pads. There are no external diaphragms at the supports. The layout of the longitudinal prestressing cables is similar to that at Choisy-le-Roi.

2.4 Two-Cell and Three-Cell Box Girder Bridges

The following bridges are of this type: Commonwealth Avenue (Australia), Lievre (Canada), Western Avenue (England), and St. Denis (France).

Commonwealth Avenue Bridge, Canberra, Australia,[10,11] is a three-cell box girder (see Fig. 11). The precast segments are 9 ft. high, 10 ft. long, and 40 ft. wide. Diaphragms occur at the piers and near the third points in each span, i.e., at the change in direction of the longitudinal prestressing cables. The girder is supported on roller bearings seated on the piers. The bridge is post-tensioned from end to end, using 1-1/8 in. diameter prestressing tendons. The cables run on either side of the four webs of the box girders and the profile is shown in Fig. 13. Vertical post-tension is applied to the webs of the box girder segments adjacent to and at the piers. Transverse post-tension is applied to all diaphragms.

The Western Avenue project, London,[49] under construction during 1968, includes two wide three-cell box girder bridges. The three-cell units, with projecting deck slabs, are cast in full-width segments. One section, shown in Fig. 15(b), has a width of 62 ft. and spans approximately 115 ft. The other section (Fig. 15(a)) is 94 ft. wide and is post-tensioned three ways: longitudinally, transversely, and vertically. The average span is 204 ft. In both cases the continuous superstructure is seated on sliding bearings on the columns.

The St. Denis Viaduct, Paris,[24] shown in Fig. 12, comprises two half bridges, each a two-cell box girder with projecting deck slab.

2.5 Discussion--Cross Section Types

The superstructures of most of the bridges described are single or multicell girders with cantilevered deck slabs or multiple single cell boxes connected by the deck slabs. The simplicity, economy, and good appearance of these sections is evident. The Hammersmith Flyover, a relatively early

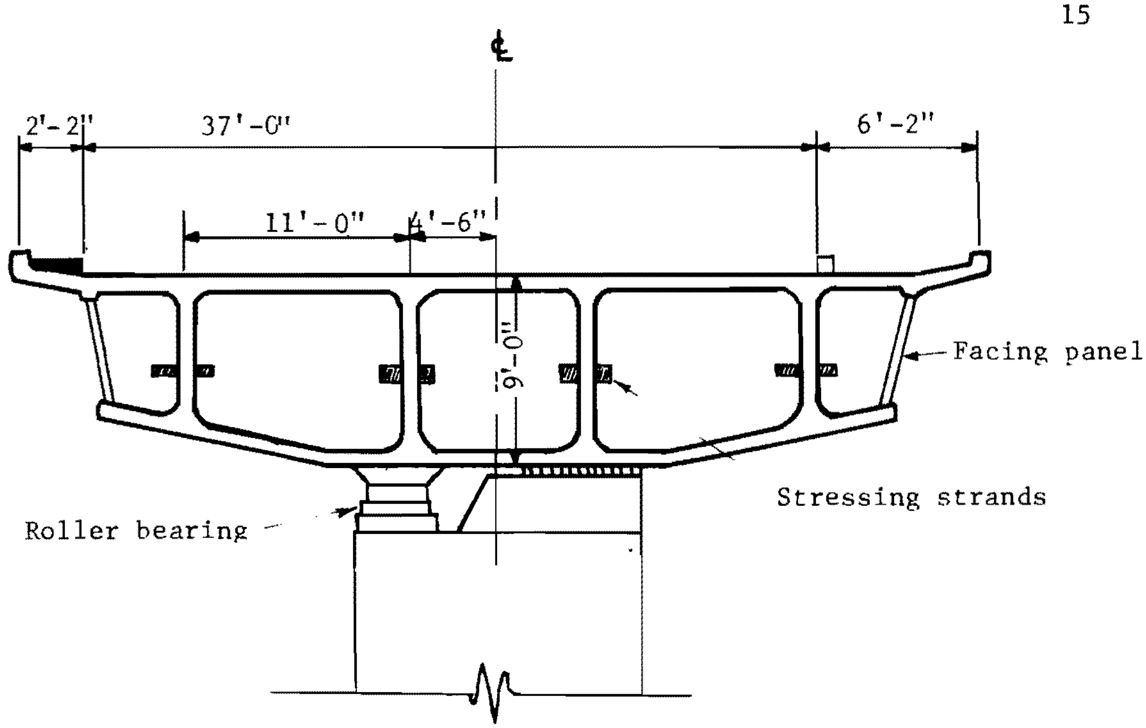


Fig. 11. Commonwealth Avenue Bridge, Canberra.

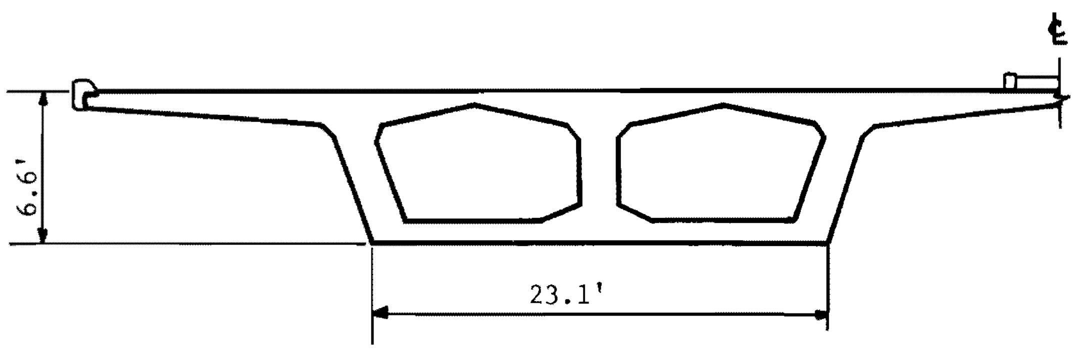


Fig. 12. Saint Denis Viaduct, Paris.

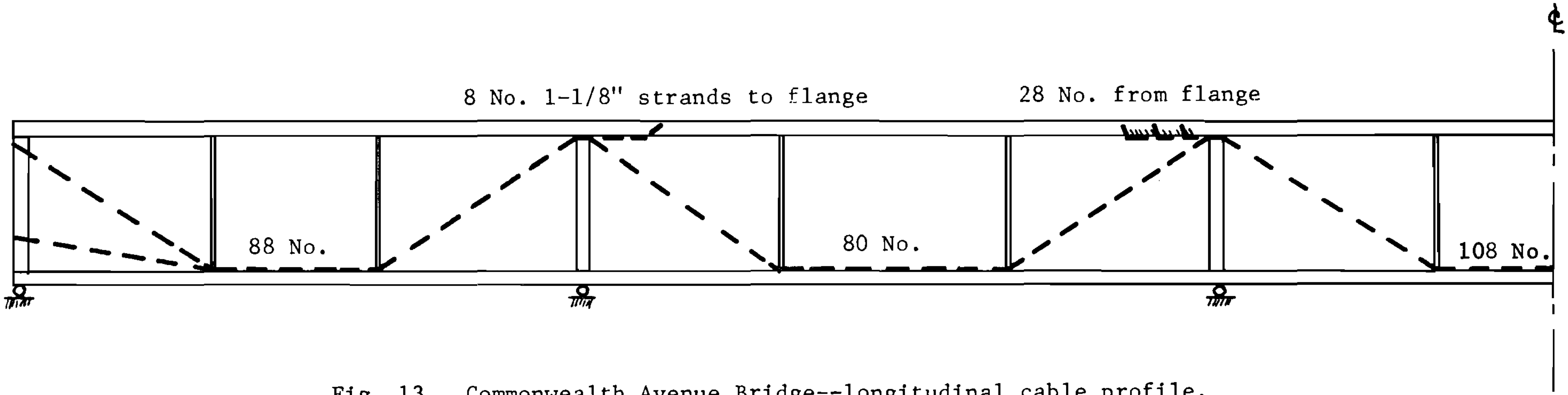


Fig. 13. Commonwealth Avenue Bridge--longitudinal cable profile.

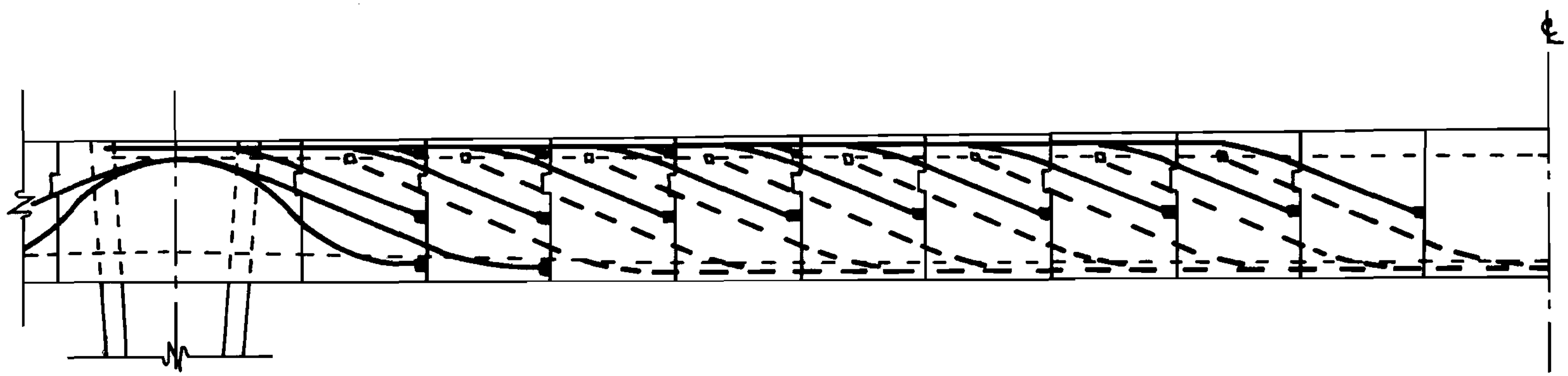
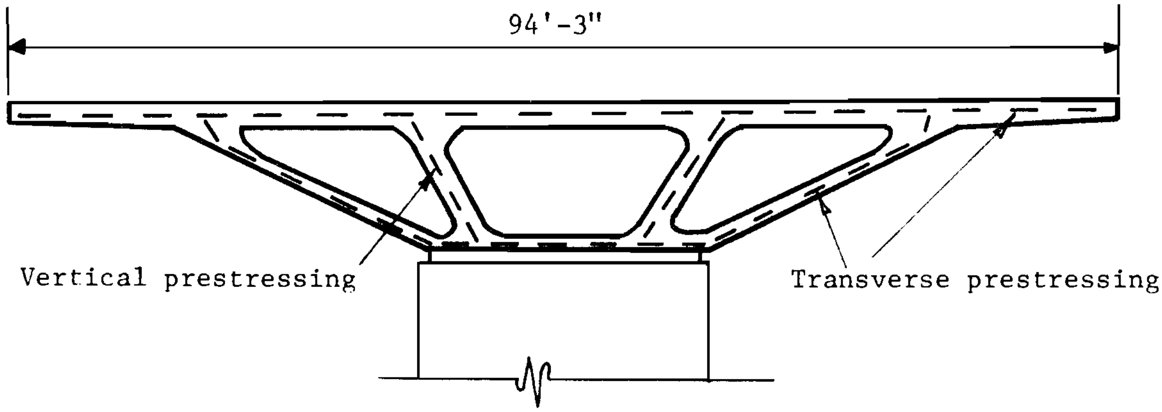
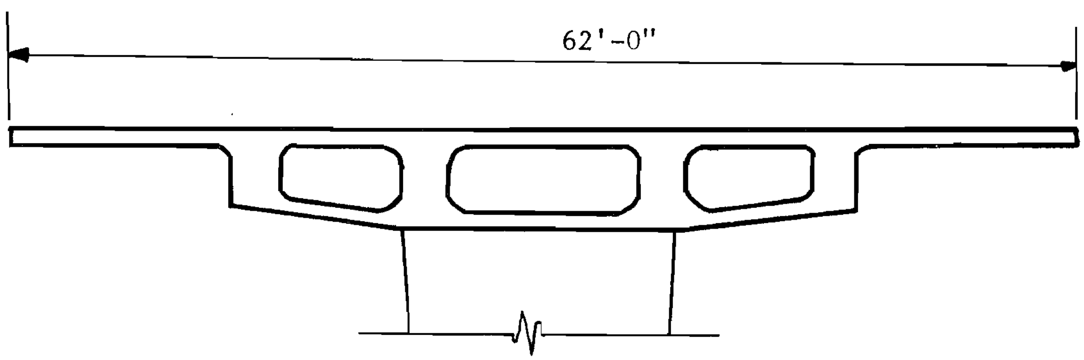


Fig. 14. Choisy-le-Roi Bridge--longitudinal cable profile.



(a) 94-ft. wide section.



(b) 62-ft. wide section.

Fig. 15. Western Avenue Viaduct, London.

bridge, is a different type. The use of the "coathanger" elements, as in this structure, would not be economical in general, because these constitute unnecessary internal diaphragms. They were used in this case partly to comply with constructional requirements.

Single cell box girders are generally used in relatively narrow bridges. As the width increases, the bending moments in the deck slab increase and hence the thickness must increase. Beyond some critical width it becomes more economical to use a multicell box or multiple single cell boxes.

In the case of multiple single cell box girders, the basic single cell units are cast separately and are connected after erection with a concrete joint and transverse post-tensioning. Thus, in general, it will be possible to have smaller basic units in this case than with a multicell box. On the other hand, a multicell box, of relatively small base width as in the Western Avenue Viaduct, may be advantageous when narrow piers are desirable. The relative economies of the two types require investigation.

2.6 Structural Systems

Most of the bridges considered are continuous over all or several spans. Exceptions are the Oosterschelde Bridge, which is a double cantilever system; Taren Point Bridge, a cantilever-suspended span system; and the Ondava Bridge, a three-span frame with a hinge in the middle of the center span.

2.7 Depth of Bridge Superstructure

Bridges having spans up to about 250 ft. are generally of uniform depth, whereas those with greater spans generally vary in depth from a maximum at the support to a minimum at midspan. The advantage of uniform depth is that all of the precast units can be the same, thus facilitating mass production. However, with variation of depth greater economy of materials is achieved. This obviously becomes more significant with increasing span.

In the case of uniform bridges the span/depth ratio is generally in the range of 20 to 27. The relation between this ratio and the cost needs investigation. Aesthetic factors are also important here; a smaller depth generally has a better appearance.

One investigation of the relation between span/depth ratio and cost is reported in Ref. 22. A three-span continuous bridge, of total length 235 ft., was considered and the cost calculated as a function of span/depth. The results are shown in Fig. 16. A ratio of 25 was found to be the most economical. However, this result, based on economic conditions in Germany, cannot be readily generalized.

2.8 Length and Weight of Precast Segments

Precast segments are generally around 10 ft. in length. This is a convenient size for transportation and erection. The weights of most segments lie in the 20 to 60 ton range.

2.9 Diaphragms

All of the bridges studied have internal diaphragms at the supports to transmit the reaction from the superstructure to the bearings and to ensure the rigidity of the box girder. In some bridges, formed from multiple single cell boxes connected by the deck slab, there are also external diaphragms to increase the rigidity of the whole superstructure. These external diaphragms, however, add to the difficulty of mass production and it may be possible to omit them altogether, as was done in the case of the Pont Aval, Paris.

Some of the earlier bridges have intermediate diaphragms between the supports. The Hammersmith Flyover has diaphragms at 10-ft. intervals and in the Commonwealth Avenue Bridge they occur at changes in direction of the prestressing cables. However, in most bridges they were found to be unnecessary and omitted.

2.10 Prestressing Systems

In bridges which run continuously over several spans and are constructed on falsework, long prestressing cables can be used. For example, in the Hammersmith Flyover and the Mancunian Way the cable length is two spans and in the Commonwealth Avenue Bridge the cables run the full length of the structure.

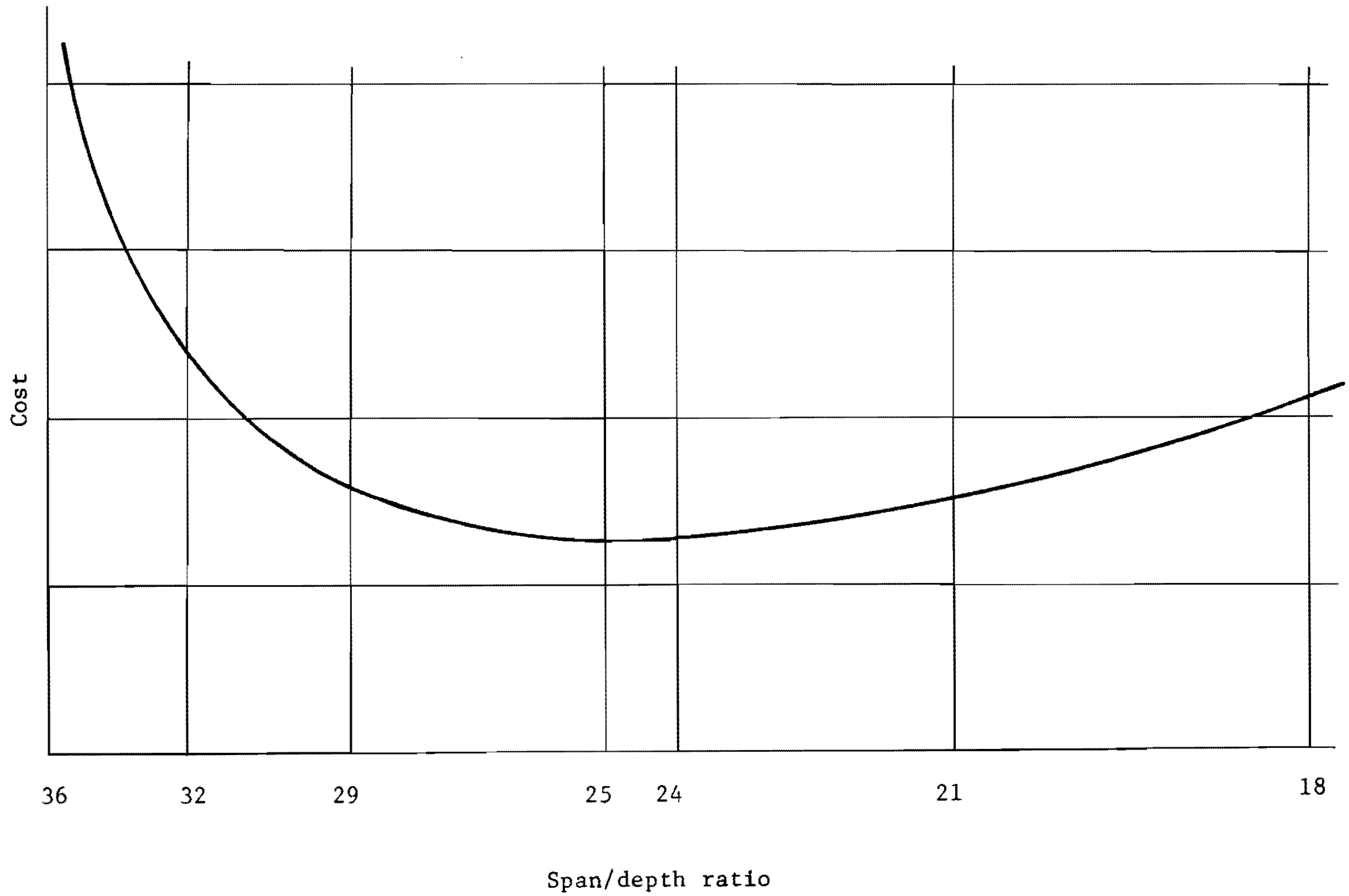


Fig. 16. Relation between cost and depth for a box girder bridge. (From Ref. 22.)

For bridges constructed by the cantilever method, the longitudinal prestressing always consists of two sets of cables, one set in the deck slab designed to resist the negative moments during construction and one set in the lower slab for positive moment near midspan.

In most bridges the prestressing cables are internal, i.e., located inside the upper and lower slabs and the webs of the box girder. However, in some cases, such as the Hammersmith Flyover, the Commonwealth Avenue Bridge, the Ager Bridge, and the Kakio Viaduct, external tendons are used. These may permit the use of smaller web thicknesses and may reduce friction losses. Shear transfer from tendons to web can be achieved by means of concrete encasement and stirrups extending from the web. External tendons do not appear to be suitable for cantilever construction or for curved cable profiles and have generally not been used in the more recent bridges.

2.11 Supports

The superstructures of the bridges are generally supported by one of the following means: (a) rigid attachment to the piers, (b) roller or sliding bearings, or (c) neoprene pads.

Neoprene pads, as used in the Pont Aval, Paris, are a very simple and economical means of support and offer possibilities for extensive use.

CHAPTER 3

SEGMENTAL CONSTRUCTION--METHODS OF PRECASTING

The general techniques of segmental construction are described by B. C. Gerwick,[12] as follows:

> Segmental construction employs precast segments cast of highest quality concrete, in sizes which can be transported and erected. . . .
>
> The segments are usually reinforced with mild steel and are designed to be connected by post-tensioning after erection. However, the segments may be prestressed in themselves. This may consist of temporary post-tensioning to aid during transportation and erection or the segments may be prestressed during their manufacture in such a manner that the prestressing is also efficient in the final position. For example, the central segment of a span can be prestressed as a simple span for transportation and erection. This initial prestressing will be additive to the connecting post-tensioning, the total providing the needed prestress at the center of the span. Segments may be pretensioned during manufacture in the transverse direction, e.g., across the deck, and later post-tensioned longitudinally after erection.

3.1 Precast Segments for the Hammersmith Flyover, London

The box girder segments and cantilever units for the Hammersmith Flyover were cast on end (i.e., on a face in contact with a joint after erection) using steel forms. Both types of units were reinforced. Six ksi concrete was used and the forms were stripped 18 hours after casting.

3.2 Segments for Glued Joint Construction

In the French bridges special techniques were developed to fabricate segments with ends suitable for epoxy resin jointing.[24,28] In order to minimize the joint thickness it was necessary to obtain a perfect fit between the mating ends of adjacent segments. This was achieved by casting each segment against the end face of the preceding one and later erecting the segments in the same order as they were cast.

Two different procedures were adopted. In the case of Choisy-le-Roi, the segments of a box girder were cast in line, using a single steel form-assembly riding on rails. The rails were set at the required profile of the underside of the bridge, allowing for camber. After each segment had set, the ends were sprayed with a bond-breaking resin and the next segment cast against it. The line of assembled units was later dismantled at the joints and reassembled in the same order during erection. An important advantage of this system is that it guarantees alignment of the span. However, the cost and difficulty increase with the span length and it is hard to provide for variation in cross section.

In the case of Pierre Benite another procedure was adopted. The forms were fixed in position and after each casting the segment was removed, first to the position of "counter-form" for the next segment and then to storage. The form-assembly included an adjustment for variation in depth. With this procedure precautions must be taken to ensure proper alignment of the fabricated bridge. In the case of the Pont Aval, which was fabricated in a generally similar manner, the alignment of each pair of segments was checked on fabrication and any error compensated for in the next segment. There was a variation in the procedure in the case of the St. Denis Viaduct, where the segments were cast on end instead of in the normal horizontal position.

In all cases the webs of the box girder were keyed at the ends of each segment to prevent vertical slip before setting of the resin. Generally, the flanges were also keyed.

3.3 Heating

Heating of the concrete is a means of accelerating the rate of setting. This has been used generally in the French precast bridges.

In the Choisy-le-Roi Bridge the segments were kept at a temperature of 113°F for six hours after casting. Two segments were fabricated per day.

In the case of Courbevoie,[40] the concrete was preheated to 86°F. During the first hour after pouring the forms were heated by insulated electric cables attached to their outer surface. One unit was cast per day.

For the Pont Aval the concrete was preheated to 104°F. The steel forms were heated, in the same manner as at Courbevoie, to maintain the concrete at this temperature as it set. Thus, setting of the concrete was accelerated from two or three days to about twelve hours.

Engineering News Record[40] reports that

> . . . the principal advantage of electrical curing over steam curing is a reduction in the maximum concrete temperature, according to the designers of Courbevoie Bridge. Steam curing can raise the temperature to 140°-160°F, which lowers the ultimate strength of the concrete. When the concrete is preheated and the forms electrically heated to 86°F, hydration raises the temperature to only 110°-120°F and the hardened concrete has a greater ultimate strength.

3.4 Discussion

Segments that will be connected by glued joints require much greater precision than those connected by concrete joints. It has been shown, with the French bridges, that this precision can be obtained by casting the segments one against the other.

The set of the concrete segments can be accelerated by heating. However, if the segments were standardized and mass-produced for several bridges, they would normally be kept in storage for some time before they were required. In this case, heating to obtain rapid set would not be necessary.

CHAPTER 4

ERECTION METHODS

Most of the bridges studied have been erected by one of the following three methods: (a) erection on falsework, (b) assembly on shore, and (c) the cantilever technique. There is, however, considerable variation in each method from bridge to bridge. Falsework may have short spans with close-spaced supports (e.g., in the case of viaducts not passing over existing roads) or else may consist of large trusses or girders, spanning from bridge pier to pier or even longer.

There is also considerable variation in lifting and placing techniques, especially with cantilever erection. Lifting devices may be supported either on or below the bridge and segments can be brought in from below or else transported along the top of the partially completed bridge.

The erection method used in each bridge is indicated in Table 1.

4.1 Erection on Falsework

A number of precast bridges, especially viaducts built over land, have been constructed on falsework.

In the case of the Hammersmith Flyover the segments were lifted onto the falsework by a gantry crane riding on rails on top of the previously placed segments. They were then adjusted in position by means of jacks. The 3-in. joints were filled with concrete and, after the design strength was reached, the longitudinal prestressing was applied. The prestressing cables were arranged longitudinally in overlapping groups, as described in Sec. 2.1, so that each newly erected span had only one group of cables in it, thus receiving initially only half the total prestressing force. The second group of cables lapped forward into the succeeding span and were inserted and stressed after erection of this span. The cables were stressed from both ends simultaneously, using Gifford-Burrow jacks, and were finally grouted in the ducts in the flange and bound to the webs with mortar casing. Lifting hooks cast into the segments were burned off after erection.

In the Mancunian Way the segments were hoisted into place on the falsework with a truck crane, utilizing temporary lifting hooks cast into the webs. The joints were concreted and the prestressing cables threaded through the ducts. Each span contained two sets of cables, each set being two spans long and overlapping the next by one span length. As each span was erected one set of cables was stressed, sufficient to take the dead load. The falsework was removed and taken forward to another span. Finally, the partially stressed span received its second set of cables and was fully stressed. The cables were tensioned from both ends at the same time.

In the Commonwealth Avenue Bridge the precast elements were lifted into position on timber falsework by an overhead traveling gantry and shifted into their exact location by hydraulic jacks. The prestressing cables, running the full length of the bridge, were tensioned from both ends. As the strands stretched several feet during stressing, the load had to be applied in stages, at each stage the strand being extended to the limit of the stressing jack, anchored temporarily and the jack moved forward and reset. The process was repeated until the desired load and elongation were obtained. After stressing the cables were encased in concrete bonded to the webs of the box girder.

In the case of the Western Avenue Viaduct a pair of large steel girders, longer than the bridge spans, were mounted on steel falsework around the columns. The tops of these girders were set at the same height as the bottom of the bridge box girder elements. Mobile cranes set the segments for one span on top of the steel girders, leaving a 4 in. gap between segments. After concreting of the joints, the prestressing cables were threaded through the ducts and the post-tension was applied.

Movable falsework trusses have been used to construct bridges over water. In the Silverwater[3,12] and Taren Point[14,15] bridges the trusses were supported on ledges at the piers. The segments were assembled on them, the 3 in. joints concreted, and the prestressing cables placed and tensioned.

A number of precast bridges in Japan have been constructed over existing highways using steel erection trusses to minimize interference with traffic. The trusses are set below, alongside, or above the bridge superstructure, depending on head-room requirements.

4.2 Assembly on Shore

Another method of erection for river bridges is to assemble the precast elements on falsework on the shore. The joints are then concreted and the prestressing applied. The assembled bridge may then be positioned by means of barges, as in the case of a number of Russian bridges,[12] or by launching over the piers, as in the Caroni Bridge in Venezuela.[5]

4.3 Cantilever Construction

A number of long-span bridges, especially those constructed over water, have been erected without falsework by the cantilever technique.[28,29] This involves placing the precast segments, two at a time, symmetrically on either side of a pier. Resistance against the increasing bending moment is provided at each stage of construction by prestressing cables of increasing length placed in the top chord of the girder.

In the case of Choisy-le-Roi the segments were transported by water and lifted into place with a floating crane. Steel framed jigs set inside the cantilever arms were used for exact positioning. As each pair of segments was placed, the prestressing cables were threaded through the ducts, the abutting faces were coated with epoxy and brought into contact and the cables were tensioned. This balanced erection procedure was continued at both piers until the symmetrical cantilever arms were completed. The gap at the center of the main span was then closed with a 16.4-ft. long closing segment. A longitudinal compressive force was applied at the joints by means of a Freyssinet flat jack, inserted at the top of the box girder webs, in order to compensate for shrinkage and prestressing shortening effects and to create an initial positive moment. The closing joints were then concreted and finally the longitudinal prestressing was completed.

Construction of the Pierre Benite Bridge was also by the cantilever method. The segments were transported by water, but in this case they were lifted into place by hoists supported on the bridge itself.

For the Pont Aval the segments were lifted by cranes supported on land or water. During cantilever erection additional temporary support for the superstructure was provided at a distance of 8 ft. from the pier centerline. This was to ensure an effective rigid fixity for the double-cantilevers, which the narrow pier top alone could not provide. After closure and post-tensioning, the girders were placed on simple neoprene supports.

The Oleron Viaduct was constructed in cantilever with the help of a 300-ft. long steel truss supported on top of the superstructure. The lower chords of the truss served as twin monorails for the suspended erection equipment. The segments were transported along the deck already constructed and lowered into place with this equipment.

A steel truss supported on the superstructure and extending over 2-1/2 spans was used in the cantilever erection of the Oosterschelde Bridge. The segments were brought in under it by barge and hoisted into place by traveling cranes mounted on the truss. The joints were concreted and prestressing applied after each pair of segments was erected.

Using the cantilever method, erection speeds of the order of 40-ft./day can be achieved with precast bridges.[24] This compares with a rate of 4-ft./day obtained in the cantilever erection of a typical cast-in-place bridge.

4.4 Discussion

Erection on falsework with close-spaced supports is the simplest method of construction when conditions permit, as in the case of viaducts over land and not passing over existing roads. Lifting and placing techniques will depend on the exact site conditions. For bridges having three or more spans over water or over existing roads, where intermediate support is not possible, the cantilever method will probably be the most suitable. There will be a critical span length, however, below which it will be more economical to use a falsework truss.

For two-span bridges over an existing highway, the two main alternatives are cantilever construction or erection on a falsework truss or girder. If cantilever construction is adopted, there are two possible procedures:

(a) to cantilever all the way from the pier to the abutments, or (b) to cantilever from the abutments as well. In the latter case, the abutments would have to be designed for this unbalanced cantilevering during erection. Erection with a falsework truss is probably more feasible than cantilevering. If necessary, the truss can be set at the same level as or above the bridge, to leave sufficient clearance for traffic on the existing road underneath. The size of the truss can be reduced if it is possible to introduce a temporary support near midspan. A third possible erection method is a combination of cantilever construction and use of a truss. The bridge can be constructed in cantilever for some distance on either side of the central pier and then completed using falsework trusses spanning from the ends of the cantilever arms to the abutments.

CHAPTER 5

JOINTS

The joints between the precast segments of a bridge are of critical importance. They must have high strength and durability and must be reasonably easy to construct. The various kinds of joints used in existing precast bridges are described in the following sections. Table 1 gives the joint type for each bridge.

5.1 Reinforced Concrete Joints

According to B. C. Gerwick:[12]

> Reinforced concrete joints 8 in. to 24 in. in width have been widely used. Reinforcing steel has been left projecting from the ends of the segments; these usually are connected by lapping or welding. High strength concrete is placed and consolidated in the joint. The achievement of high early strength is achieved by low water-cement ratios, high early strength cements, and steam curing. Steam curing of joints was employed on the Bay Bridge reconstruction, and on the Likhachev Bridge across the Moscow River. Normally, the ends of segments are constructed as vertical planes, with roughened surfaces.

5.2 Unreinforced Concrete Joints

In several of the bridges in Table 1 the precast segments were connected by cast-in-place or grouted joints of unreinforced concrete or mortar. The joint width is generally between 1 in. and 4 in., but widths up to 16 in. also occur. The end faces of the segments generally contain rectangular indentations to serve as shear keys between the precast elements and the cast-in-place concrete.

5.3 "Buttered" Mortar Joints and Dry Packed Joints

Gerwick[12] reports that "buttered joints of mortar have generally not proven successful, due to stress concentrations from inequality of mortar thickness."

"Dry packed joints of 1 in. width have been tried but it is difficult to achieve uniformly good workmanship."[12]

5.4 Epoxy Resin Joints

In a number of bridges constructed by the cantilever method, especially in France, Japan, and Russia, the precast elements were connected by a thin film of epoxy resin. A typical joint thickness is 1/32 in.[16] These epoxy joints require perfectly matching surfaces on the ends of adjacent segments, and the techniques for achieving this have been described in Sec. 3.2. Shear keys are required in the webs to transmit vertical shear forces while the resin sets.

Tests to determine the strength of epoxy resin joints have been carried out in England, France, Japan, Czechoslovakia, and Russia. The results of two series are given in Refs. 8 and 52.

Reference 52 describes a series of compressive, bending, tensile, and shear tests carried out at Sheffield and Bradford Universities. The resin used in these tests was "Dupoxy EP-018." Three compression tests on 2-7/8 in. cubes of the resin gave a minimum failure stress of 11,900 psi. Another specimen, consisting of a pair of 6 in. concrete cubes connected by a 1/16 in. resin joint, failed in the concrete at a stress of 7000 psi, while the joint remained intact. Eight specimens, consisting of a pair of 12 in. x 4 in. x 4 in. concrete blocks (f'_c = 6000 psi), connected by an epoxy joint, were tested in bending. The modulus of rupture varied from 196 psi to 409 psi. Failure occurred in the concrete or the laitance but not in the resin. Comparable results were obtained in the tensile tests. Another eight specimens, consisting of three 6 in. concrete cubes (f'_c = 6000 psi) were tested in shear. The shear stress at failure varied between 580 psi and 760 psi. Failure occurred either in the resin or in the resin and laitance.

Reference 8 describes a series of 36 shear tests on specimens comprising three concrete blocks, each 6 in. wide, 9 in. deep, and 9 in. long, and glued together to form a beam 27 in. long. The resin used was "Colma Fix" and the tests were done by the Cement and Concrete Association. The loading system was such that a point of contraflexure occurred at the joints. A uniform longitudinal compression of 100 psi was also applied in some cases. The average shear stress at failure was 680 psi in the specimens without the longitudinal force and about 100 psi higher in the others. All failures started in the concrete, although in some cases the failure ran partly along the resin-concrete boundary. The undamaged parts of nine of the specimens were tested in bending. In all cases, failure occurred in the concrete at a modulus of rupture of about 700 psi.

5.5 Dry Joints

Dry joints are those in which the segments are in direct contact. They were used in the Ojat Bridge (Russia),[32] the bridge near Bomberg (Poland), and also in California in the tunnel portion of the Bay Bridge Reconstruction Project.[12] In the latter case, chamfering of the joint edges was found desirable to prevent local spalling while stressing.

5.6 Discussion

The most widely used joints are unreinforced concrete joints and epoxy resin joints.

For bridges constructed on falsework, unreinforced concrete joints have been used in nearly all cases. They are simple to make and do not require fine tolerances in the precast segments. The joint thickness is usually between 1 in. and 4 in. Epoxy resin joints could also be used with this mode of construction, but may be less economical in view of the finer tolerances required.

For bridges erected by the cantilever method, the construction time depends largely on the rate of setting of the joints. Each pair of segments can be placed only after the joints for the previous pair have set. So with this form of erection, epoxy resin joints, which have a much faster rate of setting, have an obvious advantage over concrete joints.

Dry joints might be an alternative to epoxy resin joints. However, tolerances on the precast segments would have to be even finer than in the latter case and greater care would have to be taken during erection to achieve satisfactory results. It would be necessary to provide waterproofing for corrosion protection and to prohibit any tension in the concrete. The shear strength of the joints would be that provided by friction and the web keys only, and might be considerably less than that of other types of joints

CHAPTER 6

METHODS OF ANALYSIS

During the past three years there have been four major analytical investigations in the USA concerning cellular box girders. Each of these investigations has, however, dealt with the complete box girder, i.e., no consideration was given to segmental construction or analysis during segmental erection. Since the referenced investigations represent a major portion (if not all) of the analytical work done in this area recently, they will be reviewed briefly herein. The order in which they are presented does not necessarily indicate the order in which the work was carried out.

The first referenced investigation, conducted by A. C. Scordelis,[35] considers the analysis of simply supported box girders with or without interior shear rigid diaphragms. The analysis is carried out employing a direct stiffness approach in conjunction with a harmonic representation of loads and displacements. The stiffness equations for the structure are developed with the aid or plate equations developed by Goldberg and Leve, which are based directly upon theory of elasticity. This analysis has the advantage that a minimum number of simplifying assumptions are made; consequently the analysis is "exact" within the assumptions of the theory ot elasticity. This approach is limited by the harmonic analysis to simply supported sections and no consideration is given to the case of interior supports.

The second investigation, also conducted by Scordelis,[45] constitutes a considerable extension of the first. In this work there were developed three different methods of analysis for box girders, each with its own advantages and disadvantages.

The first method is quite similar to the analysis discussed above. The same type solution procedure is utilized and the stiffness equations for the structure are derived in an identical manner. This method, henceforth referred to as the "folded plate method," does make provisions for interior supports and diaphragms, but since a harmonic analysis is again employed, the requirement that the girder be simply supported at the two ends is still present.

The second method developed by Scordelis in this latter investigation is called the "finite segment method." The general method of attack in this case is to divide the length of the girder into a number of segments and each segment into a number of plate elements running longitudinally. The solution is accomplished by a progression down the length of the structure, satisfying equilibrium and compatibility at certain points on the edge of each element. With these equations and the boundary conditions the displacements at each division can be obtained. The primary advantage of this analysis is that it can be employed with various boundary conditions, namely fixity or simple support at interior and exterior supports. It is speculated that the computation time required for the solution would be slightly less than that required for the folded plate method. The method has the disadvantage that it is based upon simplifying assumptions of the ordinary folded plate theory rather than the more exact equations of elasticity which are the basis of the folded plate method. The numerical solution is quite sensitive; therefore, it is recommended that for a larger structural system double precision be used. Double precision would obviously increase both the required storage and computation time.

The third method of analysis employed by Scordelis is the "finite element method," utilizing a rectangular element with six degrees of freedom per nodal point. The membrane action and plate bending action of each element is considered separately and element stiffness equations for each action are derived. The element stiffness equations are then compiled into a structure stiffness equation, whereupon the analysis is carried out in an identical manner to the folded plate method, i.e., direct stiffness solution. The finite element method is the most general of the methods investigated by Scordelis in this work. The size, thickness, and material properties of each element can be varied arbitrarily throughout the structure, and arbitrary loading can be considered. The method can handle various boundary conditions by specification of displacements at the required sections. The primary disadvantage of this method of solution is the vast amount of computer storage required for a moderate size program.

Wright, Abdel-Samad, and Robinson have reported on three different methods of analysis in two different reports.[50,57] The first of the methods is, in principle, exactly like the folded plate method of Scordelis; however, it is more general in that this analysis considers stiffened plate elements.[50] The method is subject to the same limitations as the folded plate method.

The second method presented by Wright et al.[50] denoted "The Generalized Coordinate Method" is based on Vlasov's theory for thin-walled beams. The method represents an extension of the Vlasov theory in that interior flexible diaphragms, stiffened plate elements, and composite cross sections are considered. This method is limited to closed section box girders, since the torsional stiffness of open sections is neglected. This method can be employed for various boundary conditions and arbitrary loading. Wright et al., however, have limited their formulation to simple and fixed supports.

It should be noted that the methods of analysis so far considered apply to box girders whose members have constant width and, except in the case of Scordelis' third method, constant thickness also.

The third method is a procedure for predicting the transverse stresses due to cross section deformation and longitudinal stresses due to warping of the cross section.[50,57] The method is based on the similarity between the differential equations describing the response of a cellular box girder subjected to loads which cause distortion of the cross section, and those describing the response of a beam on an elastic foundation. For this reason the method is termed the "BEF analogy." This analogy results in a convenient method to design the dimensions and spacing of diaphragms, given the other girder parameters. The method is quite general in that it can be used for nonprismatic sections, girders continuous over interior supports, and girders with flexible or rigid interior diaphragms. The method is of limited usefulness, since it must be used in conjunction with some other method of analysis in order to determine the "other" plate parameters mentioned above. It is a quite useful design tool and has been shown to be accurate enough for design purposes. Wright has developed an approximate design procedure based on the BEF analogy which is seen to yield results accurate enough for design. However, in some cases the method is quite conservative. This design procedure has the same advantages and limitations as the BEF analogy for predicting transverse distortion stresses.

Mattock and Johnston have conducted investigations aimed at correlating analytical results with results obtained from model tests.[54] The analytical procedure used was quite similar to the folded plate method developed by Scordelis and has the same advantages and limitations as the first method discussed herein.

From the above discussion it is evident that although some work has been done concerning box girders with various end conditions, diaphragm situations, stiffness properties, etc., there is no information available regarding work done on the problem of segmentally constructed box girders. Considerations such as longitudinal and/or transverse prestress, computation of losses, jointing, and stresses during erection will not, of course, change the problem at hand completely; it will, however, add several additional complications heretofore not considered.

CHAPTER 7

CONCLUSIONS

It is clear from the numerous bridges already constructed that segmentally precast box girders provide an effective means of achieving long spans.

A variety of box girder shapes and prestressing systems have been used. The most suitable cross-sectional shapes for bridges of 30 ft.-50 ft. width appear to be a one or two-cell box or a pair of single cells connected by the deck slab.

Efficient casting techniques have been developed to obtain the tolerances required, both for concrete jointing and for epoxy resin jointing. In the latter case the required precision is obtained by casting the segments one against the other.

The most frequent methods of construction are erection on falsework and cantilever erection. The former is the simpler method if the temporary supports can be set at close intervals. Otherwise, cantilever erection or a combination of both methods may be more suitable.

In most of the precast bridges considered, joints have been either of concrete or of epoxy resin. Both types have proved satisfactory. For cantilever erection, epoxy resin joints are superior, as they allow a greater speed of construction. Dry joints have been rarely used and there is insufficient information available to evaluate them.

There is need for some study of the relative advantages and economy of box girders and possible alternative structural elements. It is proposed in Ref. 48 that continuous spans up to about 200 ft. may be obtained by combining AASHO Type VI I-girders with a haunched girder or inclined piers. Bulb T-girders, another alternative, have been designed by Abam Engineers, Inc.

(Tacoma, Washington), for simple spans up to 160 ft. and for slightly greater spans in continuous bridges. However, one great advantage of precast box girders is that they can be used for a much larger range of spans, including spans of 300 ft. as required for stream crossings.

Detailed comparative studies of precast and cast-in-place box girder bridges are not available. However, some advantages of precasting are clear. Better quality control is possible and greater speed of erection. With standardization and mass production it is probable that precasting will have a significant economic advantage over cast-in-place construction.

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A P P E N D I X

BRIDGE AND LOCATION	YEAR	FIG.	SPANS ft	NO OF BOXES	CELLS PER BOX	b_u ft	b_l ft	d_{max} ft	d_{min} ft	$\frac{L}{d_{min}}$	SEGMENT LENGTH ft	JOINTS	ERECTION METHOD	SUPPORT SYSTEM AND SPECIAL FEATURES
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
AUSTRALIA														
SILVERWATER BRIDGE, SYDNEY	1962		120+200+120	2	1							C 3	F T	DECK CAST IN PLACE.
COMMONWEALTH AVENUE BRIDGE, CANBERRA	1963	2.8	185+210+240+210+185	1	3	40	40	9	9	27	10	C 3	F	ROLLER BEARINGS ON PIERS.
TAREN POINT BRIDGE, SYDNEY	1964		+(2x235)+250+(2x235)+	4	1	21	11		10.5	24	9	C 3	F T	CANTILEVER-SUSPENDED SPAN SYSTEM. BOX SECTION AND SLAB CAST SEPARATELY.
AUSTRIA														
TRAUN BRIDGE, LINZ	1961		+246+308+246+	4	1	22	79	13.4	13.4	23	13	C	F	
AGEN BRIDGE	1963		238+278+278+117	1	1	46	17.7	14	14	22	30	C	F	
CANADA														
LIEVRE RIVER BRIDGE, QUEBEC	1967		130+260+130	1	2	33	22	14.6	5.8	—	9.5	E	C	BRIDGE AND SEGMENTS ARE SKEWED.
CZECHOSLOVAKIA														
ONDAVA BRIDGE, SIRNIK	1964		98+197+98	2	1	18	7.9	10	4.6	--	9.8	C 1.2	C	STRUCTURE FORMS A 3-SPAN FRAME WITH A HINGE IN THE MIDDLE. DECK SLAB BETWEEN BOXES IS CAST SEPARATELY.
KOSICKE HAMRE BRIDGE	1965		126+252+126								9.8	C 1.2	C	
RAILWAY BRIDGE, MARGECANY	1966		100+180+100	1	1	18.4		15	7.1	25	9.8	C	C	
ENGLAND														
HAMMERSMITH FLYOVER, LONDON	1961	2.2	+(11x140)+	1	3	61	10	9	6.5	21	10	C 3	F	GIRDERS RIGIDLY CONNECTED TO COLUMNS. SUPERSTRUCTURE INCLUDES BOX SECTION AND "COATHANGER" UNITS.
MANCUNIAN WAY, MANCHESTER	1966	2.4	+(28x105)+	1	1	30	9	4.3	4.5	25	7.5	C 3	F	"ROTAFLON" SLIDING BEARINGS ON COLUMNS.
WESTERN AVENUE VIADUCT, LONDON	1968	2.11	SEVERAL x 115 (APPROX.) SEVERAL x 204 (APPROX.)	1 1 1	3 3 3	62 94		5.5 10.25	5.5 10.25	21 20	7.7 7.8	C 4 C 4	F G F G	SLIDING BEARINGS ON COLUMNS.
FRANCE														
CHOISY-LE-ROI BRIDGE, PARIS	1965	2.6	123+180+123	2	1	22	12	8.2	8.2	22	8.2	E	C	SUPERSTRUCTURE FORMS A PORTAL WITH THE PIERS AND IS SIMPLY-SUPPORTED ON THE ABUTMENTS.
BRIDGE ON THE RHONE AT PIERRE BENITE	1965		184+276+184 164+246+246+164	2 2	1	27	11.5	14	11.8	23	9.8	E	C	GIRDERS ARE MADE CONTINUOUS WITH THE CAISSONS.
OLERON VIADUCT	1965	2.5	+(26x259)+	1	1	35	18	14.8	8.2	--	10.8	E	C	SEMIRIGID SUPPORT FROM NEOPRENE PADS ON THE PIERS.
COURBEVOIE BRIDGE, PARIS	1966		131+197+131	4	1	29		7.5	7.5	26	12.5	E	C	
ST DENIS VIADUCT, PARIS	1966		+(8x157)+	2	2	47	23.1	6.6	6.6	24	9.8	E	F	DECK SLAB PROJECTIONS CAST SEPARATELY. BRIDGE IS CURVED.
PONT AVAL, PARIS	1967	2.7	221+301+287+234	2	1	28	11.5	18	11.1	29	12.0	E	C	SIMPLE SUPPORT FROM NEOPRENE PADS ON THE PIERS.
JAPAN														
KAKIO VIADUCT	1964		69+(3x122)	1	1	25	13.1	6.0	6.0	20	10.6	C 14	F	
KAMIOSAKI VIADUCT	1966		88+102+88 75+129+75	2	1	24	9.8	4.9	4.9	26	8.2	E	C	
KONOSHIMA BRIDGE	1968		138+282+138	1	1	27	13.1	14.1	6.6	--	9.8	E	C	
TAMA BRIDGE	1969		164+165+165+164	2	1	20		8.8	8.8	19	11	E	C	
MEXICO														
BRIDGE IN VERACRUZ STATE	1965		20+131+20	1	1	14	7	7.2	7.2	18	5	C	C	
NETHERLANDS														
OOSTERSCHELDE BRIDGE	1965	1.1	(52x312)	1	1	39	22	19	7	--	42	C 16	C	GIRDERS CONTINUOUS WITH PIERS, DOWELLED EXPANSION JOINTS AT SPAN CENTERS.
HARTELKANAAL BRIDGE	1967		220+375+220	2	1	28		17.3	4.9	--				
POLAND														
BRIDGE NEAR BOMBERO	1966		59+141+59								9.8	D	C	
RUSSIA														
AUTOSAWODBRIDG	1960		69+487+173									C 8	C	
OJAT BRIDGE	1961		105+210+105								7.2	D	C	
IRTYSCH BRIDGE	1961		+361+	3							10.7	C 0.8	C	
SCHELNICHA BRIDGE, MOSKO	1964		198+420+198			48					9.8	E	C	
DON BRIDGE, ROSTOW	1964		260+455+260								9.8	E	C	
VENEZUELA														
CARONI BRIDGE			157+(4x315)+157	1	1	34	13.1	18.4	18.4	17			[illegible]	