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

Segmental prestressed concrete bridges are described in terms of cross-sectional shape, layout of tendons, casting procedures, jointing techniques, and design details. Techniques of analysis for structures of this type are reviewed. Prospects for use of this type of construction in the United States are discussed and current work outlined.

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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 esthetics.

The February 1967 report[38] 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." To comply with these safety criteria, two-span, three-span, and four-span overpasses will be required with spans up to 180 ft. (55 m).

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. (90 m) range will sometimes be necessary. When all factors are considered the trend toward long span bridges seems irreversible.

Precast I-girders, of the type now widely used for spans up to 120 ft. (37 m), will not be adequate for these longer spans. It was proposed[48] that AASHO Type VI I-girders might be used for simple spans up to 140 ft. (43 m) and continuous spans up to 160 ft. (49 m) (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.

Table 1. Segmental precast, post-tensioned box girder bridges

Bridge and Location	Year	Spans ft.	No. of Boxes	Cells Per Box	b ft.	b' ft.
Australia						
Silverwater Bridge, Sydney	1962	120—200—120	2	1	—	—
Commonwealth Avenue Bridge, Canberra	1963	185—210—240—210—185	1	3	40	40
Taren Point Bridge, Sydney	1964	2 @ 235—250—2 @ 235	4	1	21	11
Austria						
Traun Bridge, Linz	1961	246—308—246	4	1	22	7.9
Ager Bridge	1963	238—278—278—117	1	1	46	17.7
Canada						
Lievre River Bridge, Quebec	1967	130—260—130	1	2	33	22
Czechoslovakia						
Ondava Bridge, Sirnik	1964	98—197—98	2	1	18	7.9
Kosicke Hamre Bridge	1965	126—252—126	—	—		
Railway Bridge, Margecany	1966	100—180—100	1	1	18.4	—
England						
Hammersmith Flyover, London	1961	11 @ 140	1	3	61	10
Mancunian Way, Manchester	1966	28 @ 105	1	1	30	9
Western Avenue Viaduct, London	1968	Several @ 115 (approx.)	1	3	62	—
		Several @ 204 (approx.)	1	3	94	—
France						
Choisy-Le-Roi Bridge, Paris	1965	123—180—123	2	1	22	12
Bridge on the Rhone at Pierre Benite	1965	184—276—184— 164—246—246—164	2	1	27	11.5
Oleron Viaduct	1965	26 @ 259	1	1	35	18
Courbevoie Bridge, Paris	1966	131—197—131	4	1	29	—
St. Denis Viaduct, Paris	1966	6 @ 157	2	2	47	23.1
Pont Aval, Paris	1967	221—301—267—234	2	1	26	11.5
Japan						
Kakio Viaduct	1964	69—3 @ 122	1	1	23	13.1
Kamiosaki Viaduct	1966	88—102—88— 75—129—75	2	1	24	9.8
Konoshima Bridge	1968	138—282—138	1	1	27	13.1
Tama Bridge	1969	164—165—165—164	2	1	20	—
Mexico						
Bridge in Veracruz State	1965	20—131—20	1	1	14	7
Netherlands						
Oosterschelde Bridge	1965	52 @ 312	1	1	39	22
Hartelkanaal Bridge	1967	220—375—220	2	1	28	—
Poland						
Bridge near Bomberg	1966	59—141—59	—	—	—	—
Russia						
Autosawodbridg	1960	119—487—119	—	—	—	—
Ojat Bridge	1961	105—210—105	—	—	—	—
Irtysch Bridge	1961	361	3	—	—	—
Schelnicha Bridge, Moscow	1964	198—420—198	—	—	48	—
Don Bridge, Rostow	1964	260—455—260	—	—	—	—
Venezuela						
Caroni Bridge	—	157—4 @ 315—157	1	1	34	13.1

Notes: See Fig. 1 for dimension designations in columns 6 to 9.
1. C3 = 3 in. concrete joint; E = epoxy joint; D = dry joint.
2. FT = falsework trusses; F = falsework; FG = falsework girders; C = cantilever construction; A = assembly on shore.

Table 1. (Cont.)

$d_{max.}$ ft.	$d_{min.}$ ft.	Span/depth ratio $\frac{L}{d_{min.}}$	Segment Length ft.	Joints[1]	Erection Method[2]	Support System and Special Features
—	—	—	—	C 3	F T	Deck cast in place
9	9	27	10	C 3	F	Roller bearings on piers
—	10.5	24	9	C 3	F T	Cantilever-suspended span system—box section and slab cast separately
13.4	13.4	23	13	C	F	
14	14	22	30	C	F	
14.8	5.8	—	9.5	E	C	Bridge and segments are skewed
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 cast separately
—	—	—	9.8	C 1.2	C	
13	7.1	25	9.8	C	C	
9	6.5	21	10	C 3	F	Girders rigidly connected to columns —superstructure includes box section and "coathanger" units
4.3	4.3	25	7.5	C 3	F	"Rotaflon" sliding bearings on columns
5.5	5.5	21	7.7	C 4	F G	Sliding bearings on columns
10.25	10.25	20	7.8	C 4	F G	
8.2	8.2	22	8.2	E	C	Superstructure forms a portal with the piers and is simply supported on the abutments
14	11.8	23	9.8	E	C	Girders are made continuous with the caissons
14.8	8.2	—	10.8	E	C	Semirigid support from neoprene pads on the piers
7.5	7.5	26	12.5	E	C	
6.6	6.6	24	9.8	E	F	Deck slab projections cast separately —bridge is curved—simple support from neoprene pads on the piers
18	11.1	29	12.5	E	C	
6.0	6.0	20	10.6	C 14	F	
4.9	4.9	26	8.2	E	C	
14.1	6.6	—	9.8	E	C	
8.8	8.8	19	11	E	C	
7.2	7.2	18	3	C	C	
19	7	—	42	C 16	C	Girders continuous with piers—dowelled expansion joints at span centers
17.3	4.9	—	—	—	—	
—	—	—	9.8	D	C	
—	—	—	—	C 8	C	
—	—	—	7.2	D	C	
—	—	—	10.7	C 0.8	C	
—	—	—	9.8	E	C	
—	—	—	9.8	E	C	
18.4	18.4	17	—	—	A	

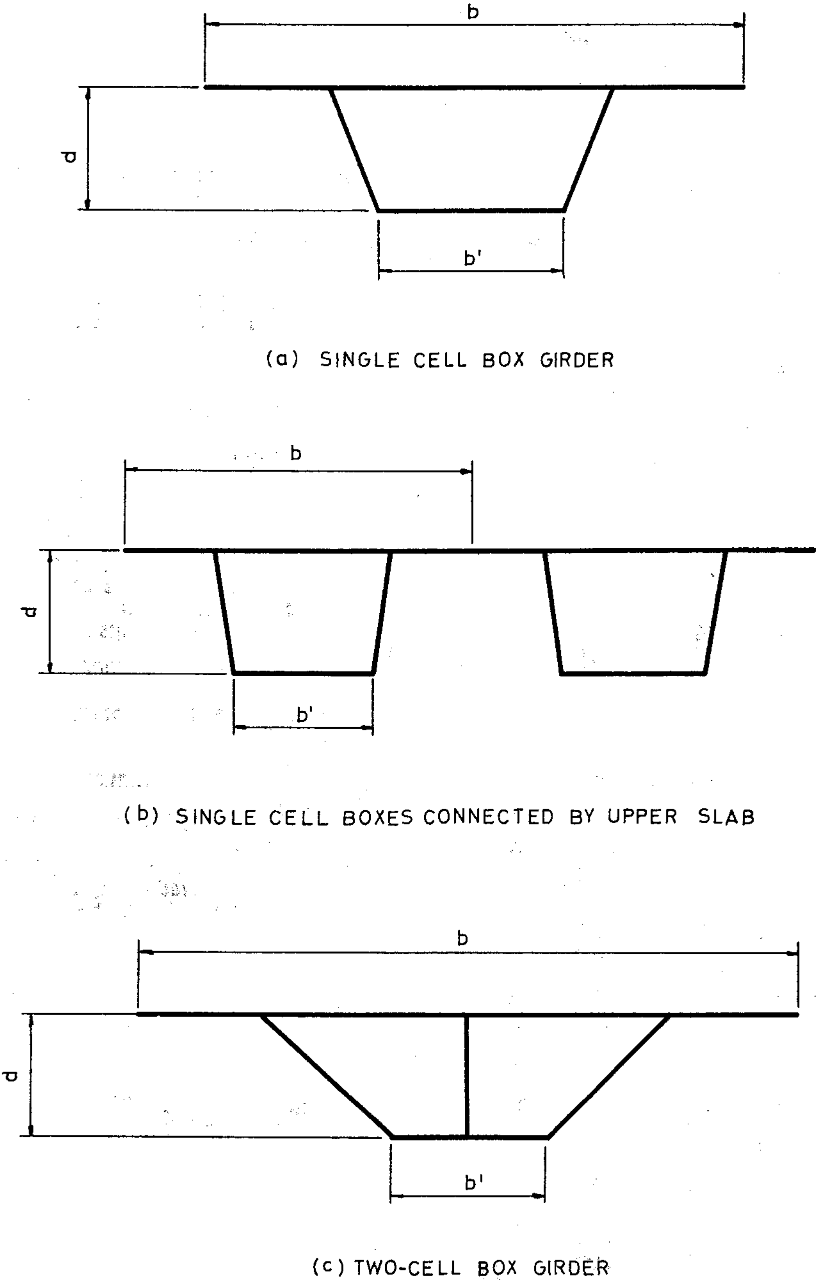

Fig. 1. Types of segmental box girder cross sections

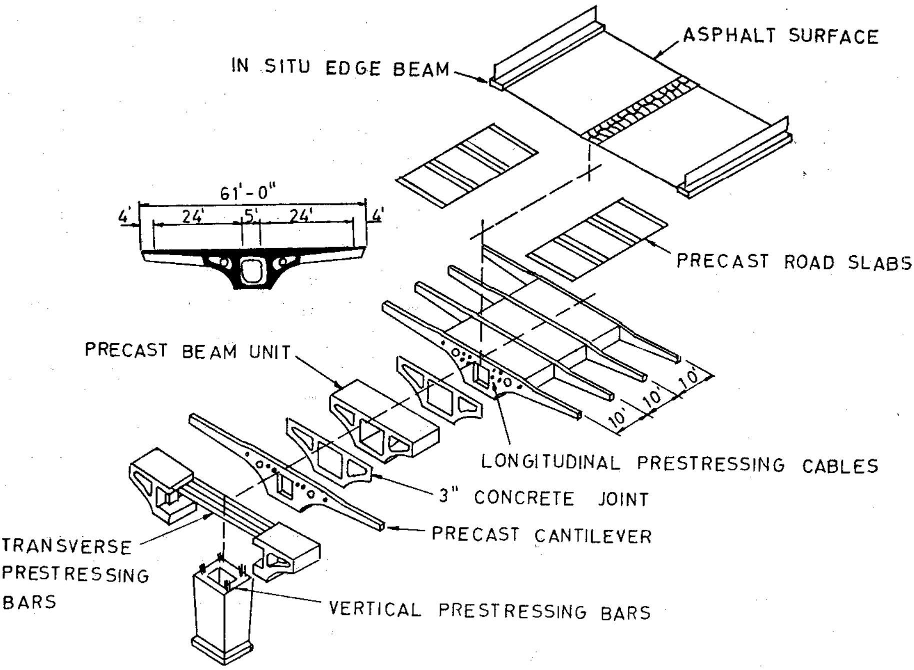

Fig. 2. Hammersmith Flyover, London, England

A substantial number of long span bridges have been constructed throughout the world utilizing prestressed concrete box girders. This type results in a very compact structural member, which combines high flexural strength with high torsional strength. The box girders may be either cast-in-place or precast in segments. Their suitability for 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. (208 m). In the United States, cast-in-place box girder bridges are being widely used by the California Division of Highways, as well as by several other states; precast, segmental box girder bridges are now under construction in Nova Scotia and Texas.

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:

1. Mass production of standardized girder units is possible, such as is presently done 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 girder units at a plant rather than casting them in place.
4. Speed of erection can be much greater which is very important when construction interferes with existing traffic and is par-

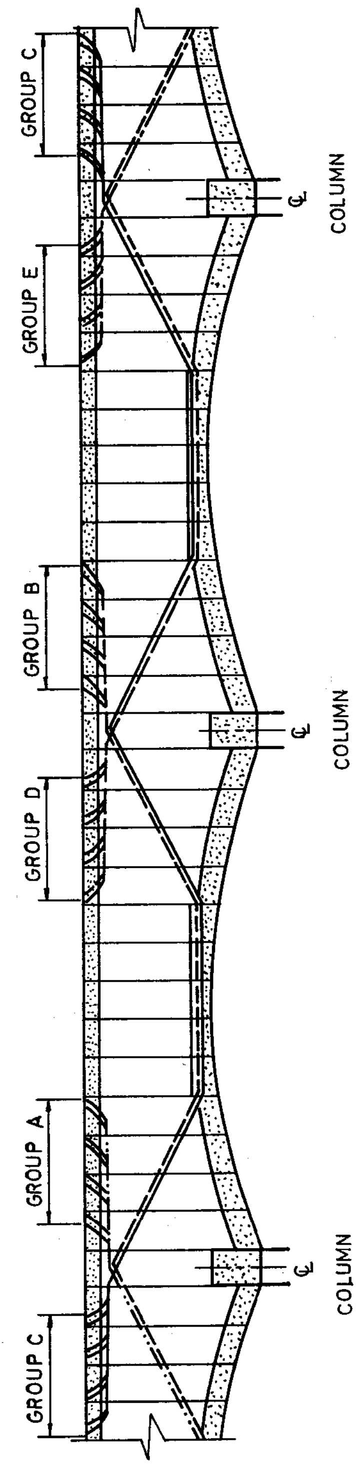

Fig. 3. Typical cable profile for Hammersmith Flyover

ticularly critical in an urban environment.

This paper will examine in detail one means of achieving long spans in bridge structures, namely, segmental precast box girder construction. The reason for casting short segments is 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 to form the completed structure. The length and weight of the segments are chosen so as to be most suitable for transportation and erection.

TYPES OF CROSS SECTION AND PRESTRESSING SYSTEMS

Dimensions and details of a number of segmental, prestressed concrete box girder bridges are summarized in Table 1 and the cross sections of many of them 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: 1. the single cell box girder; 2. a pair of single cell box girders connected by a deck slab; 3. the multi-cell box girder (see Fig. 1). 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.

Hammersmith Flyover. The Hammersmith Flyover in London[1,2,6] is of unique construction (see Fig. 2). The main girder element is a three-cell hollow box, 26 ft. (7.9 m) wide, cast in segments 8 ft. 6 in. (2.6 m) long. These alternate with 1 ft. (0.3 m) thick, precast cantilever "coat hanger" sections 60 ft. (18.3 m) wide.

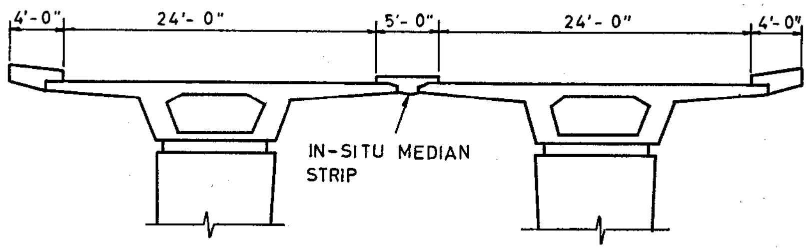

Fig. 4. Mancunian Way, Manchester, England

The sections are joined by 3 in. (7.6 cm) 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 at their bases.

The Gifford-Udall system is used for longitudinal prestressing. Stranded cables, 1⅛-in. (2.86 cm) in diameter, are arranged in four clusters of 16 each, one cluster on either side of each inner web of the girder. At midspan, each cluster passes through a 10-in. (25.4 cm) 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. 3). 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. (7.6 m) on either side of midspan; cable profiles are linear between saddles.

The girder segments over the columns are prestressed transversely through the top flange and are tied to the columns with vertical prestressing.

Single cell box girders. The following bridges are of this type: Ager (Austria), Margecany (Czechoslova-

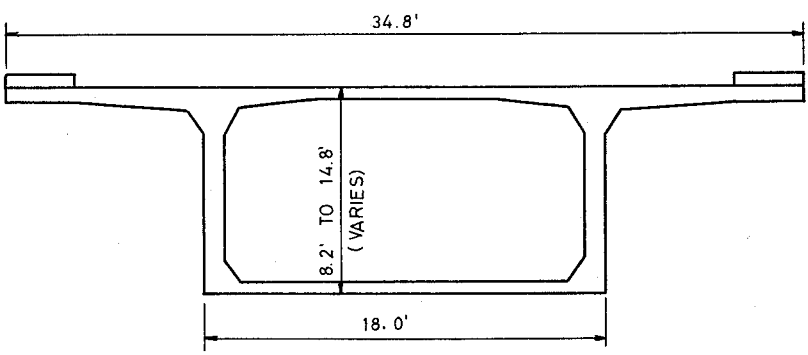

Fig. 5. Oleron Viaduct, France

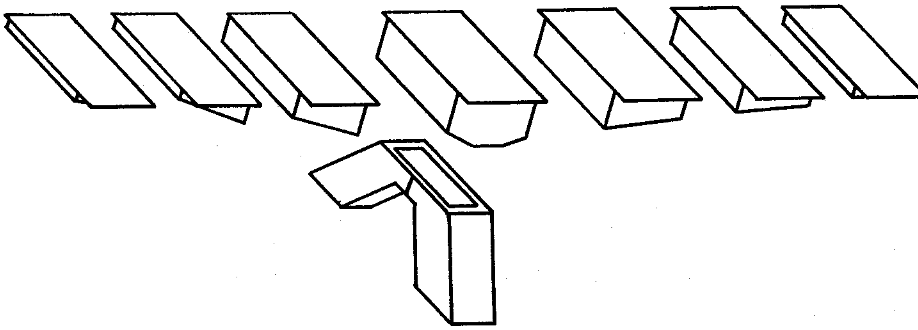

Fig. 6. Superstructure of the Oosterschelde Bridge, Netherlands

kia), 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. 4). The upper slab of each box is cantilevered to accommodate the roadway width. The precast segments are 7 ft. 3 in. (2.2 m) long, with 3-in. (7.6 cm) concrete joints between. The vertical webs of the box girders are made thick enough to accommodate all the prestressing cables. There are 16 cables through each section with two layers of four in each web. Each span contains two sets of cables, each set being two spans long, overlapping the next set by one span length, and anchored near the quarter points of the spans. The girders are supported by Rotaflon sliding bearings on the columns.

Oleron Viaduct in France[20,24,42] is a single cell box girder bridge, with the upper deck slab cantilevered out to a total width of 35 ft. (10.7 m) (see Fig. 5). The precast seg-

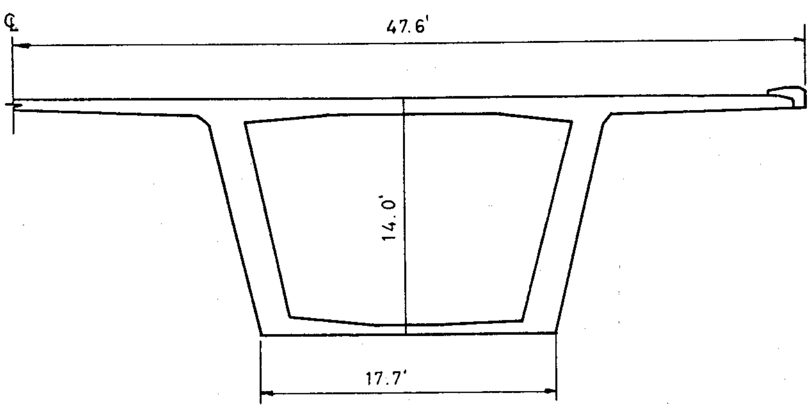

Fig. 7. Ager Bridge, Austria

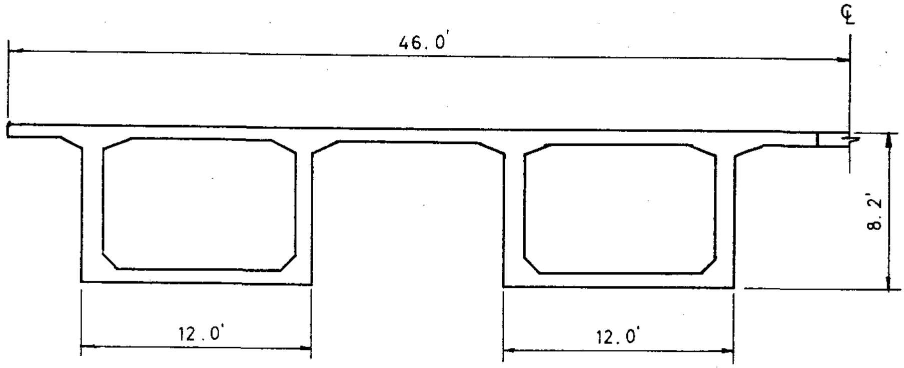

Fig. 8. Choisy-le-Roi Bridge, Paris, France

ments are 10.8 ft (3.3 m) long. The girders are prestressed in both the longitudinal and transverse directions and are 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. 6). The precast segments are about 41 ft. (12.5 m) long and vary in depth from 19 ft. (5.8 m) at the piers to 7 ft. (2.1 m) at midspan. 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 with a few in the lower slab near midspan. 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 made up of precast segments 30 ft. (9.2 m) long with cantilevered upper slabs and a constant depth of 14 ft. (4.3 m). The prestressing cables are external to the section within the cell but bonded to the web by concrete encasement and held by stirrups anchored in the web.

Parallel single cell boxes connected by a deck 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, France[16,33], consists of two identical half bridges (see Fig. 8). These are connected by a 4 ft. (1.2 m) wide concrete slab with a longitudinal hinge joint on either side, and are tied together with transverse post-tensioning. The three-span continuous girders form a portal frame with the two piers and are simply supported at the abutments.

Each half bridge, 44 ft. (13.4 m) in width, consists of two box girders, 8.2 ft. (2.5 m) in depth and cast separately in 8.2 ft. long segments. The top slab of each box is cantilevered

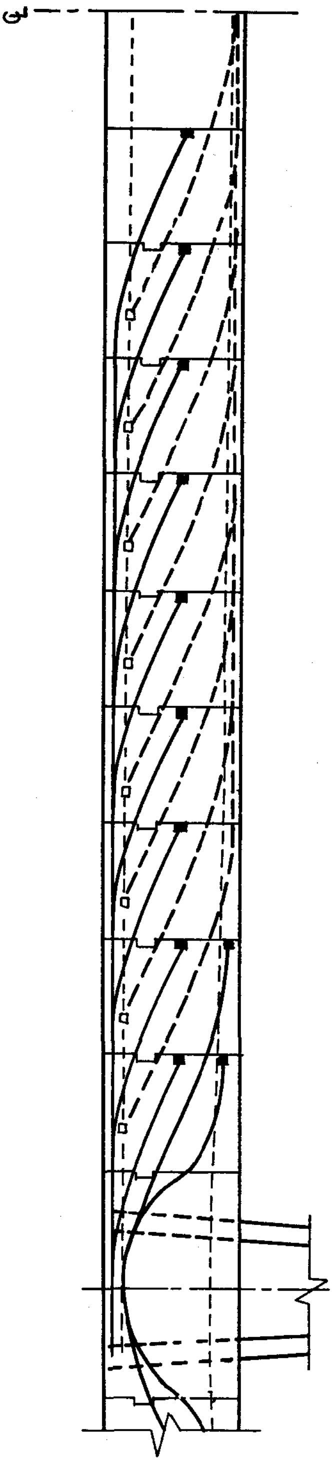

Fig. 9. Longitudinal cable profile for Choisy-le-Roi Bridge

out in both directions. After erection and longitudinal post-tensioning, the two girders 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 and externally (i.e., between the two box girders) but no intermediate diaphragms.

There are two sets of longitudinal prestressing cables (see Fig. 9). 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 second set of cables is located in the lower slab for positive moment near midspan with most of them draped and anchored in the deck slab.

The bridge on the Rhone at Pierre Benite[34] (Fig. 10) consists of a pair of single cell box girders with projecting upper slabs, rigidly connected after erection. The depth is 11.8 ft. (3.6 m), except near the supports where it increases to approximately 14 ft. (4.3 m). Each box is cast in segments 9.8 ft. (3.0 m) 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. Two diaphragms at each caisson are extended to form a transverse box section bracing the two box girders. There are single diaphragms at the abutments but no intermediate diaphragms. The girder segments and the diaphragms at the caissons and abutments are cast in

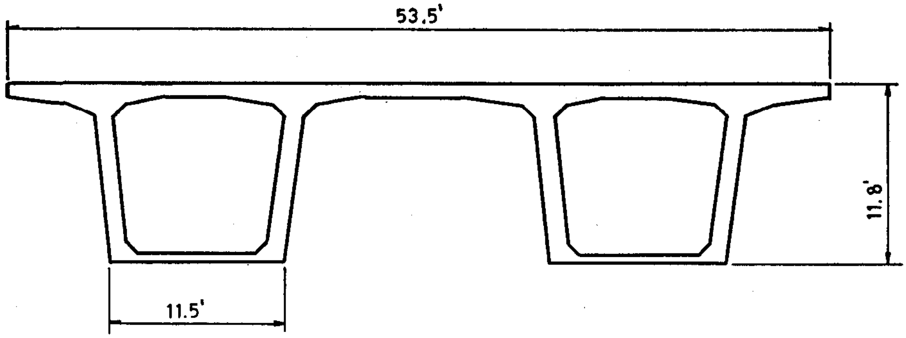

Fig. 10. Rhone River bridge at Pierre Benite, France

place, because of the skew of the bridge, whereas all other segments are prefabricated.

As in the case of the Choisy-le-Roi Bridge, 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. (1.1 m) wide slab and transverse post-tensioning is applied.

The superstructure of the Pont Aval in Paris[41,52] is similar to that of Pierre Benite (see Fig. 11). It consists of a pair of single cell box girders, with upper slabs rigidly connected by an 8-in. (20.3 cm) cast-in-place joint and transverse post-tensioning. Overall width is 52 ft. (15.9 m); girder depths are 11.1 ft. (3.4 m) over the central portions increasing to 18 ft. (5.5 m) at the supports. The precast segments are 12.5 ft. (3.8 m) long. The bridge has four continuous spans and is supported on neoprene pads. The layout of the longitudinal prestressing cables is similar to that of the Choisy-le-Roi Bridge.

Multi-cell box girders. The following bridges are of this type: Common-

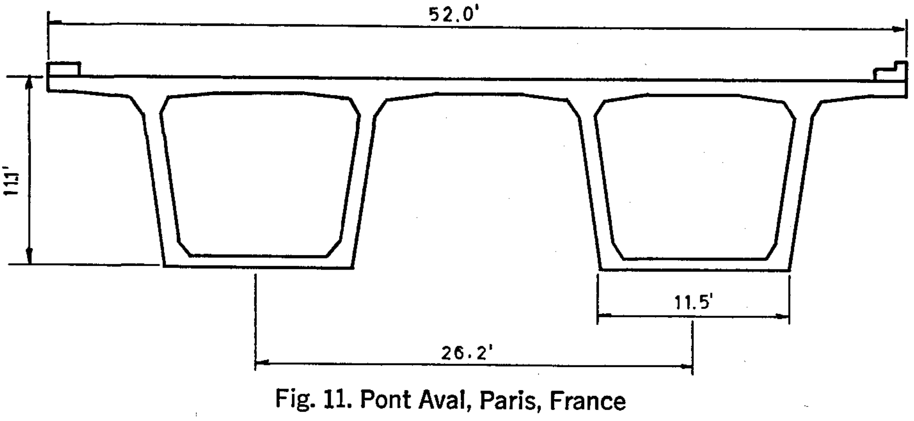

Fig. 11. Pont Aval, Paris, France

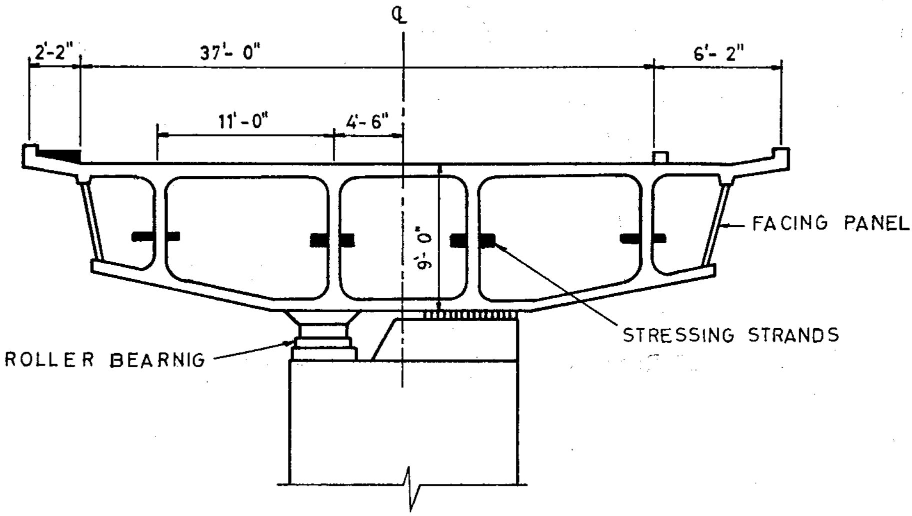

Fig. 12. Commonwealth Avenue Bridge, Canberra, Australia

wealth Avenue (Australia), Lievre (Canada), Western Avenue (England), and St. Denis (France).

Commonwealth Avenue Bridge, Canberra, Australia(10,11), is a three-cell box girder consisting of precast segments 9 ft. (2.7 m) high, 10 ft. (3.0 m) long, and 40 ft. (12.2 m) wide (see Fig. 12). Diaphragms are included at the piers and near the third points in each span, where a change in direction of the longitudinal prestressing cables occurs. The girder is supported on roller bearings seated on the piers. The bridge is post-tensioned from end to end, using 1⅛-in. (2.86 cm) diameter tendons placed on either side of the four webs of the box girders. The tendon profile is shown in Fig. 13. Vertical post-tensioning was applied to the webs of the box girder segments adjacent to and at the piers and transverse post-tensioning to all diaphragms.

The Western Avenue Viaduct in London(49), includes two different wide, three-cell box girder bridges. Both three-cell units, with projecting deck slabs, were cast in full-width segments (see Fig. 14). One unit has a width of 62 ft. (18.9 m) and the girder spans approximately 115 ft. (35.0 m). The other unit is 94 ft. (28.7 m) wide and is post-tensioned longitudinally, transversely and vertically; the average girder span is 204 ft. (62 m). In both cases the continuous superstructure is seated on sliding bearings on the columns.

The St. Denis Viaduct, Paris, France(24), shown in Fig. 15, comprises two half bridges, each a two-cell box girder with projecting deck slab.

METHODS OF PRECASTING

The methods of precasting used with segmental box girder bridges will be largely dependent on the procedures selected for erection and jointing. General precasting advantages have been outlined by Gerwick(12). Precasting operations generally follow normal procedures and segments have been cast in many different types of forms and positions.

The box girder segments and cantilever units for the Hammersmith Flyover were cast on end (i.e., on a face that later would be in contact with a cast-in-place joint after erection) using steel forms. Both types of units contained normal reinforcement and 6000-psi (420 kg/cm^2) concrete, and they were stripped 18 hours after casting.

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 they were cast.

For the Choisy-le-Roi Bridge, the segments of a box girder were cast in line, using a single steel form assembly riding on rails. The rails were set to 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 then dismantled 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 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

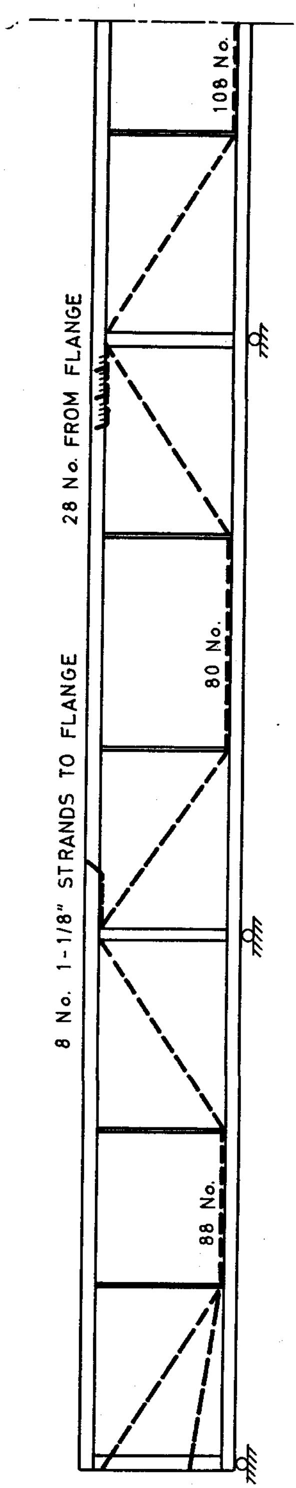

Fig. 13. Longitudinal cable profile for Commonwealth Avenue Bridge

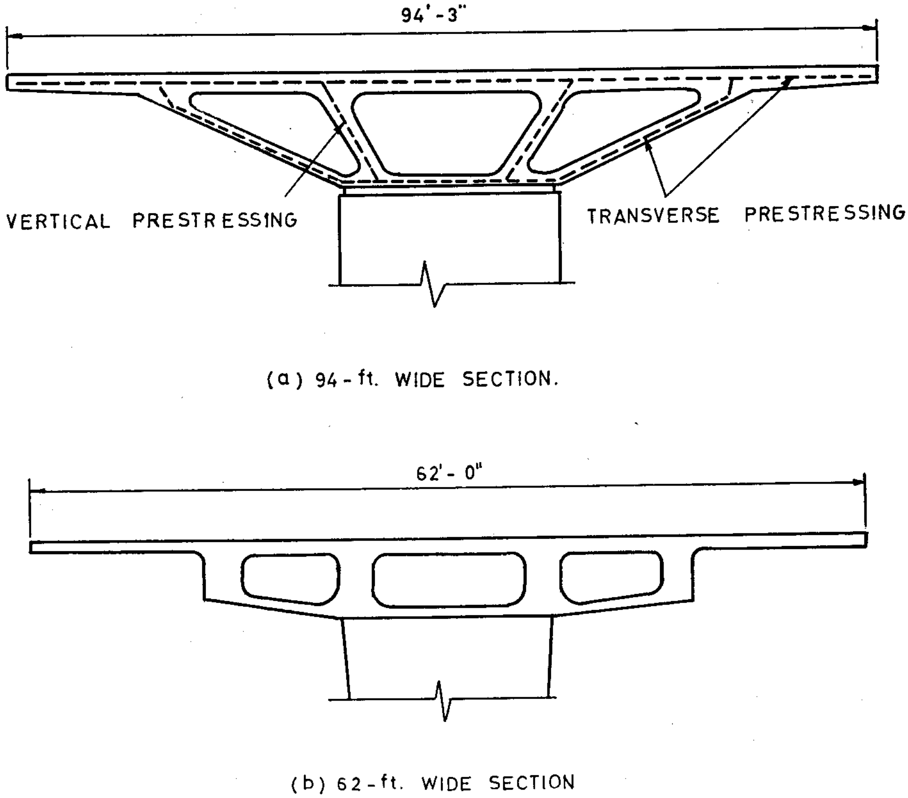

Fig. 14. Western Avenue Viaduct, London, England

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 girder. Pont Aval was fabricated in a similar manner with the alignment of each pair of segments checked on fabrication and any error compensated for in the next segment. The segments for the St. Denis Viaduct were cast on end instead of in the normal horizontal position.

In all cases the ends of the webs of the box girder segments were keyed to prevent vertical slip before setting of the epoxy. Generally, the flanges were also keyed.

ERECTION METHODS

Most of the bridges studied have been erected by one of the following three methods: 1. erection on falsework, 2. assembly on shore, and 3. the cantilever technique. There may be considerable variation in each method from bridge to bridge. For example, falsework may have short spans with closely spaced supports, as in the case of a viaduct not passing over existing roads, or may consist of large trusses or girders spanning from bridge pier to bridge 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; segments can be brought in from below or transported along the top of the partially completed bridge.

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

Erection on falsework. A number of segmental precast bridges, especially viaducts built over land, have been constructed on falsework. On 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. (7.6 cm) joints were filled with concrete and, after design strength was reached, the longitudinal prestressing was applied. The prestressing cables were arranged longitudinally in overlapping groups, as described on page 59, 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 was inserted and stressed after erection of this span. All cables were stressed from both ends simultaneously, 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.

On 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 sufficiently to take the dead load. 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 again tensioned from both ends at the same time.

On the Commonwealth Avenue Bridge, the precast elements were lifted into position on timber falsework by an overhead traveling gantry crane and shifted into their exact location by hydraulic jacks. The prestressing cables, running the full

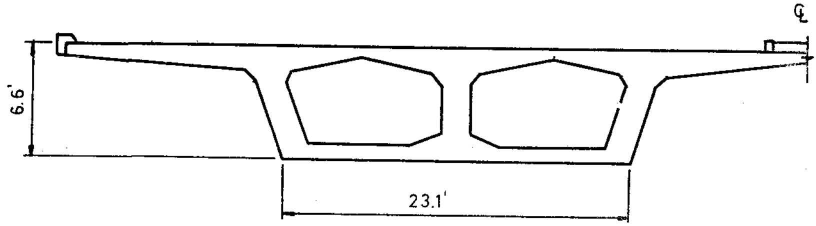

Fig. 15. Saint Denis Viaduct, Paris, France

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, each stage being limited by the stroke of the jack, anchored temporarily, and the jack moved forward and reset. 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, was mounted on steel falsework around the columns. The tops of these girders were set at the same height as the bottom of the box girder elements. Mobile cranes set the segments for one span on top of the steel girders, leaving 4-in. (10.2 cm) gaps between segments. After concreting of the joints, the prestressing cables were threaded through the ducts and post-tensioned.

Movable falsework trusses have been used to construct bridges over water. In the Silverwater[3,12] and Taren Point[14,15] bridges in Australia, the trusses were supported on ledges at the piers. The segments were assembled on them, the 3-in. (7.6 cm) joints concreted, the prestressing cables placed and tensioned, and the trusses moved forward. A number of 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.

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 girders 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 on the Caroni Bridge in Venezuela[5].

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 negative bending moment is provided at each stage of construction by adding prestressing cables of increasing length in the top chord of the girder. The pier must be designed for possible unbalanced moment loadings or supplemental struts or ties may be provided.

In the case of the Choisy-le-Roi Bridge, the segments were transported by water and lifted into place with a floating crane. Steel 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. (5.0 m) 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.

On the Pierre Benite Bridge, also by cantilever construction, the segments were transported by water and lifted into place by hoists supported on the bridge itself.

For Pont Aval, the segments were lifted by cranes on both land and water. During cantilever erection, additional temporary support for the girders was provided at a distance of 8 ft. (2.4 m) 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 with the help of a 300 ft. (91 m) 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 2½ 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 construction method, erection speeds of the order of 40 ft. (12 m) per day can be achieved with precast segmental bridges[24]. This compares with a rate of 4 ft. (1.2 m) per day using cantilever erection for a typical cast-in-place bridge.

JOINTS

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

Concrete joints. Reinforced concrete joints 8 in. to 24 in. (20.3 to 60.9 cm) in width have been widely used. Reinforcing steel left projecting from the ends of the segments are usually connected by lapping or welding. High strength concrete is placed and consolidated in the joint[12].

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. (2.5 and 10.0 cm), but widths up to 16 in. (40.7 cm) have been used. 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.

Gerwick reports[12] 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. (2.5 cm) width have been tried but it is difficult to achieve uniformly good workmanship."

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 layer of epoxy resin no greater than $\frac{1}{32}$-in. (0.8 mm)[16]. These epoxy joints require perfectly matching surfaces on the ends of adjacent segments, achieved by match casting. Shear keys in the webs transmit vertical shear forces while the resin sets and also serve a very useful function in controlling alignment.

Tests to determine the strength of thin epoxy resin joints have been carried out in England, France, Japan, Czechoslovakia and Russia[8,52]. These tests have indicated that it is possible to develop 94 percent of the flexural tensile strength of a comparative monolithically cast test specimen and approximately 75 percent of the shear strength. Failures generally occur in the concrete but not in the epoxy. The type of epoxy resin must be compatible with damp surface conditions and have a minimum pot life of 1½ hours.

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

METHODS OF ANALYSIS

The past half-decade has seen a proliferation of publications dealing with the analysis of complete plate assemblages, i.e., folded plates and box girders[63]. All of these procedures are applicable to completed bridges and do not consider segmental construction problems. The most generally applicable analytical methods can be categorized as: 1. folded plate analysis, 2. thin-walled beam theory, and 3. finite element techniques.

Folded plate analysis is the most exact of the three approaches. The behavior of the plate element in its plane is governed by the plane stress equations of elasticity, while that normal to the plane is governed by the equations of biaxial plate bending. The Goldberg-Leve load displacement equations were utilized, in conjunction with the direct stiffness technique, by Scordelis[35,45] to develop a general purpose computer program for the analysis of continuous, prismatic box girders of arbitrary cross section. Intermediate and support diaphragms were considered. This method has also been utilized by Mattock and Johnston[54] to compare analytical results with those from an extensive model analysis with reported good correlation.

A majority of the analytical techniques fall into the thin-walled beam theory category. This approach takes advantage of the fact that, for the type of structures under consideration, longitudinal moments and associated slab shears are quite small and can, consequently, be neglected.

Wright, et al[50], have extended the "generalized coordinate" technique of box girder analysis, originally presented by Vlasov, to account for anisotropic plate elements, flexible interior diaphragms and intermediate supports. The method makes further assumptions on the deformation of the basic plate element by neglecting torsional moments and shear distortion. However, the agreement reported between this method and a more refined analysis is good.

Lo and Scordelis[58] have adopted the assumption that longitudinal and torsional moments are negligible, to arrive at still another formulation falling into the thin-walled beam theory category. The technique, termed "the finite segment method", requires division of the structure both transversely and longitudinally into rectangular sub-regions. Each sub-region is assumed to behave as a beam in the longitudinal direction and a one-way slab in the transverse direction. The primary advantage is the generality of boundary and loading conditions which may be treated. Restraint from boundary and interior supports is specified independently for each element of the cross section; consequently, the structure may be completely free, fixed, or something in-between at the boundaries and supports. Both intermediate, shear-rigid diaphragms and diaphragms over supports may be handled. The noted assumptions result in the minimum number of degrees of freedom necessary to model the behavior of a box girder structure. However, they also inflict limitations upon the method, which can produce inaccuracies in the results under certain circumstances. Scordelis(45) reports that correlation between the approximate finite segment method and the more exact folded plate method is good.

The finite element method is the most general of the analytical techniques. The structure is divided into sub-regions, the deformation patterns of which are assumed. Displacement functions are assumed such that when displacement compatibility is explicitly enforced at two element nodal points, the displacements along the element interface connecting the two points will be compatible. Once the displacement assumptions are made, the element stiffness matrix may be derived, and the analysis is carried out by the direct stiffness method.

Scordelis(45) and Meyer and Scordelis(62) have employed the finite element method for the analysis of box girder structures. Scordelis employs a rectangular element with six degrees of freedom per nodal point. The size, thickness, and material properties of the elements may be varied arbitrarily throughout the structure, and arbitrary loading may be considered. Numerous boundary conditions, and both intermediate, shear-rigid diaphragms and diaphragms over supports, may be treated. The work of Meyer and Scordelis extends this analysis to include consideration of plate stiffeners and flexible frame supports. The primary disadvantage associated with the finite element method is the number of equilibrium equations which must be solved, requiring vast computer storage and extended execution times for even moderate size problems. Scordelis(45) has compared results of the finite element method with those of the folded plate method and reports good correlation. As the finite element mesh size decreases the correlation improves indicating convergence of his finite element analysis.

The analytical techniques discussed above involve a large number of computations and, therefore, necessitate use of a digital computer.

Wright, et al(57), and Tung(61) have used a technique for analysis of single cell, rectangular or trapezoidal, box girders which is suitable for hand calculation. The analysis falls into the thin-walled beam theory category. The analysis for the torsional load component is based upon the analogy between the differential equations governing the response of a beam on an elastic foundation and that governing the response of the single cell to the distortional load component. The technique is quite general in that it may be applied to continuous, prismatic or haunched girders with arbitrary end conditions, anisotropic plate properties, and interior rigid or flexible diaphragms. The disadvantage is that it

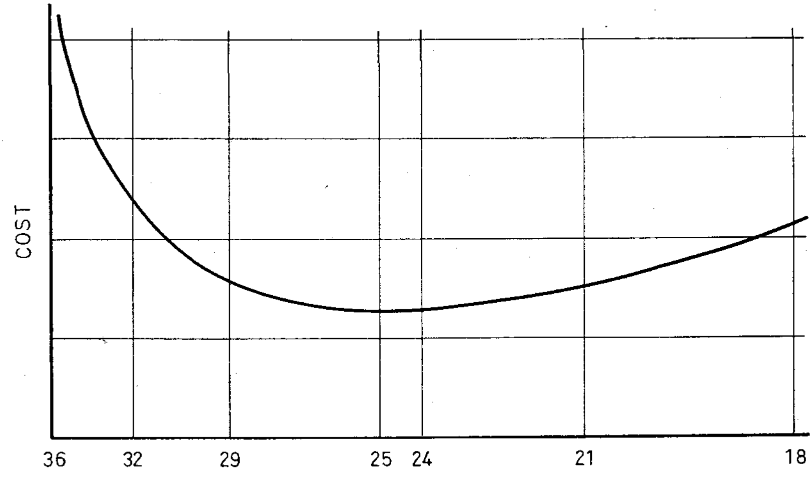

Fig. 16. Relation between span/depth ratio and cost

is strictly applicable to single cell sections. Wright compares the results from the beam on elastic foundation analogy with results from a more rigorous analysis and concludes that this method predicts the longitudinal and transverse stresses resulting from torsional load components adequately for design purposes.

It is evident that although considerable analytical capability is available for analysis of completed box girder structures, there has been little consideration given the analysis of segmentally constructed prestressed girders. Considerations such as longitudinal and/or transverse prestressing, and the continuously changing structural system to be analyzed, do not, of course, alter the basic problem. They do, however, add several additional complications heretofore not considered.

A research effort presently underway at The University of Texas is incorporating capabilities for considering arbitrary longitudinal prestressing and segmental stage construction into the finite segment analysis. These capabilities, coupled with those already a part of the analysis, will result in a general purpose computer program for the analysis of segmentally constructed prestressed box girders at each stage of erection.

SUMMARY AND CONCLUSIONS

Most of the bridges which have been segmentally constructed 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.

Bridges having spans up to about

250 ft. (75 m) 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. In the case of uniform depth bridges, the span/depth ratio is generally in the range of 20 to 27. The relation between this ratio and cost needs investigation. Esthetic factors are also important here since a smaller depth generally has a better appearance.

One investigation of the relation between span/depth ratio and cost[22] is of a three-span continuous bridge with a total length of 235 ft. (72 m). The results, shown in Fig. 16, indicate a span/depth ratio of 25 to be the most economical. However, this result, based on economic conditions in Germany, cannot be readily generalized.

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. External diaphragms have been found to add to the difficulty of mass production and it may be possible to omit them altogether, as was done with Pont Aval, Paris. Some of the earlier bridges have intermediate diaphragms between supports, however, in most bridges they were found to be unnecessary and they are omitted.

In bridges which are continuous over several spans and are constructed on falsework, long prestressing cables for two or more spans can be used. 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. External tendons permit the use of smaller web thicknesses and may reduce friction losses, but they 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.

The superstructures of the bridges are generally supported by means of rigid attachment to the piers, roller or sliding bearings, or neoprene pads. Neoprene pads, as used in Pont Aval, are a very simple and economical means of support and offer possibilities for extensive use.

Erection on falsework with close-spaced supports is the simplest method of construction when conditions permit. For bridges having three or more spans 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: 1. to cantilever all the way from the pier to the abutments, or 2. 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. A third possibility 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.

The most widely used joints are unreinforced concrete and epoxy resin. For bridges constructed on falsework, unreinforced concrete joints have been used in nearly all cases. The 1 in. to 4 in. (2.5 to 10.0 cm) thick joints are simple to make and do not require exacting tolerances in the precast segments. For bridges erected by the cantilever method, construction time depends largely on the rate of setting of the joints and epoxy resin joints, which have a much faster rate of setting, have an obvious advantage over concrete joints. Dry joints, as an alternative to epoxy resin joints, leave much to be desired and have not been used extensively.

A one-sixth scale model of a prototype segmentally constructed bridge to be built near Corpus Christi, Texas, is currently being tested under a contract with the Texas Highway Department. The completed bridge, of uniform 10 ft. (3.0 m) depth, will span the Intercoastal Canal with a 200 ft. (61 m) main span flanked by two 100 ft. (30.5 m) outer spans. The construction of the scale model has allowed a careful study of forming techniques and match casting for thin epoxy joints. Results from the model test will be compared with the predicted performance from analysis.

There is need to study the relative advantages and economy of box girders and possible alternative structural elements. Some work along these lines has been done at The University of Texas at Austin in studying costs and developing an optimization technique. It is proposed[48] that continuous spans up to about 200 ft. (61 m) 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 for simple spans up to 160 ft. (49 m) 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. (91 m) as required for stream crossings.

Although detailed comparative studies of precast and cast-in-place box girder bridges are not available, the factors of standardization and mass production, better quality control and greater speed of erection indicate that precasting may have a significant economic advantage over cast-in-place construction.

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