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11

Segmental Concrete Bridges

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11.1 Introduction

Before the advent of segmental construction, concrete bridges would often be made of several precast girders placed side by side, with joints between girders being parallel to the longitudinal axis of the bridge. With the modern segmental concept, the segments are slices of a structural element between joints which are perpendicular to the longitudinal axis of the structure.

When segmental construction first appeared in the early 1950s, it was either cast in place as used in Germany by Finsterwalder et al., or precast as used in France by Eugène Freyssinet and Jean Muller. The development of modern segmental construction is intertwined with the development of balanced cantilever construction.

By the use of the term *balanced cantilever construction,* we are describing a phased construction of a bridge superstructure. The construction starts from the piers cantilevering out to both sides in such a way that each phase is tied to the previous ones by post-tensioning tendons, incorporated into the permanent structure, so that each phase serves as a construction base for the following one.

The first attempts to use balanced cantilever construction, in its pure form, were made by Baumgart, who in 1929 built the Río Peixe Bridge in Brazil in reinforced concrete, casting the 68-m-long main span in free cantilevering. The method did not really prosper, however, until the post-tensioning technique had been sufficiently developed and generally recognized to allow crack-free concrete cantilever construction.

From 1950, several large bridges were built in Germany with the use of balanced cantilever construction with a hinge at midspan, using cast-in-place segments, such as

- Moselbrücke Koblenz, 1954: Road bridge, 20 m wide, with three spans of 101, 114, 123 m plus short ballasted end spans hidden in large abutments; the cross section is made up of twin boxes of variable depth, connected by the top slab.
- Rheinbrücke Bendorf, 1964: Twin motorway bridges, 1,031 m long, with three main river spans of 71, 208, 71 m, built-in free cantilever construction with variable depth box sections.

In France, the cantilever construction took a different direction, emphasizing the use of precast segments.

Precast segments were used by Eugène Freyssinet for construction of the well-known six bridges over the Marne River in France (1946 to 1950). The longitudinal frames were assembled from precast segments, which were prestressed vertically and connected by dry-packed joints and longitudinal post-tensioning tendons. Precast segments were also used by Jean Muller for the execution of a girder bridge in upstate New York, where longitudinal girders were precast in three segments each, which were assembled by dry-packed joints and longitudinal post-tensioning tendons.

From 1960, Jean Muller systematically applied precast segments to cantilever construction of bridges. It is characteristic for precast segmental construction, in its purest form, that segments are match cast, which means that each segment is cast against the previous one so that the end face of one segment will be an imprint of the neighbor segment, ensuring a perfect fit at the erection. The early milestones were as follows:

- Bridge over the Seine at Choisy-le-Roi in France, 1962: Length 37+55+37 = 130 m; the bridge is continuous at midspan, with glued joints between segments (first precast segmental bridge).
- Viaduc d'Oleron in France, 1964 to 1966: Total length 2862 m, span lengths generally 79 m, with hinges in the quarterpoint of every fourth span; the segments were cast on a long bench (long-line method); erection was by self-launching overhead gantry (first large-scale, industrialized precast bridge construction).

In the same period, precast segmental construction was adopted by other designers for bridge construction with cast-in-place joints. Some outstanding structures deserve mention:

- Ager Brücke in Austria, 1959 to 1962: Precast segments placed on scaffold, cast-in-place joints.
- Río Caroni in Venezuela, 1962 to 1964: Bridge with multiple spans of 96-m each. Precast segments 9.2 m long, were connected by 0.40-m-wide cast-in-place joints to constitute the 480-m-long bridge deck weighing 8400 tons, which was placed by incremental launching with temporary intermediate supports.
- Oosterschelde Bridge in The Netherlands, 1962 to 1965: Precast segmental bridge with a total length of 5 km and span lengths of 95 m; the precast segments are connected by cast-in-place, 0.4-m-wide joints and longitudinal post-tensioning.

Since the 1960s, the construction method has undergone refinements, and it has been developed further to cover many special cases, such as progressive construction of cantilever bridges, span-by-span construction of simply supported or continuous spans, and precast-segmental construction of frames, arches, and cable-stayed bridge decks.

In 1980, precast segmental construction was applied to the Long Key and Seven Mile Bridges in the Florida Keys in the United States. The Long Key Bridge has 100 spans of 36 m each, with continuity in groups of eight spans. The Seven Mile Bridge has 270 spans of 42 m each with continuity in groups of seven spans. The spans were assembled from 5.6-m-long precast segments placed on erection girders and made self-supporting by the stressing of longitudinal post-tensioning tendons. The construction method became what is now known as span-by-span construction.

Comparing cast-in-place segmental construction with precast segmental construction, the following features come to mind:

- Cast-in-place segmental construction is a relatively slow construction method. The work is performed *in situ,* i.e., exposed to weather conditions. The time-dependent deformations of the concrete become very important as a result of early loading of the young concrete. This method requires a relatively low degree of investment (travelers).
- Precast segmental construction is a fast construction method determined by the time required for the erection. The major part of the work is performed in the precasting yard, where it can be protected against inclement weather. Precasting can start simultaneously with the foundation work. The time-dependent deformations of the concrete become less important, as the concrete may have reached a higher age by the time the segments are placed in the structure. This method requires relatively important investments in precasting yard, molds, lifting gear, transportation, and erection equipment. Therefore, this method requires a certain volume of work to become economically viable. Typically, the industrialized execution of the structure leads to higher quality of the finished product.

Since the 1960s, the precast segmental construction method has won widespread recognition and is used extensively throughout the world. Currently, very comprehensive bridge schemes, with more than 20,000 segments in one scheme, are being built as large urban and suburban viaducts for road or rail. It is reasonable to expect that the precast segmental construction method, as introduced by Jean Muller, will contribute extensively to meet the infrastructure needs of humankind well into the next millennium.

11.2 Balanced Cantilever Girder Bridges

11.2.1 Overview

Balanced cantilever segmental construction for concrete box-girder bridges has long been recognized as one of the most efficient methods of building bridges without the need for falsework. This method has great advantages over other forms of construction in urban areas where temporary shoring

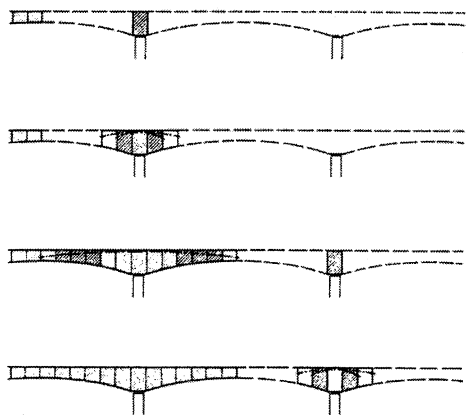


FIGURE 11.1 Balanced cantilever construction.

would disrupt traffic and services below, in deep gorges, and over waterways where falsework would not only be expensive but also a hazard. Construction commences from the permanent piers and proceeds in a "balanced" manner to midspan (see Figure 11.1). A final closure joint connects cantilevers from adjacent piers. The structure is hence self-supporting at all stages. Nominal out-of-balance forces due to loads on the cantilever can be resisted by several methods where any temporary equipment is reusable from pier to pier.

The most common methods are as follows:

- Monolithic connection to the pier if one is present for the final structure;
- Permanent, if present, or temporary double bearings and vertical temporary post-tensioning;
- A simple prop/tie down to the permanent pile cap;
- A prop against an overhead gantry if one is mobilized for placing segments or supporting formwork.

The cantilevers are usually constructed in 3- to 6-m-long segments. These segments may be cast in place or precast in a nearby purpose-built yard, transported to the specific piers by land, water, or on the completed viaduct, and erected into place. Both methods have merit depending on the specific application.

It is usually difficult to justify the capital outlay for the molds, casting yard, and erection equipment required for precast segmental construction in a project with a deck area of less than 5000 m^2. The precasting technique may be viable for smaller projects provided existing casting yard and molds can be mobilized and the segments could be erected by a crane.

11.2.2 Span Arrangement and Typical Cross Sections

Typical internal span-to-depth ratios for constant-depth girders are between 18 and 22. However, box girders shallower than 2 m in depth introduce practical difficulties for stressing operations inside the box and girders shallower than 1.5 m become very difficult to form. This sets a minimum economical span for this type of construction of 25 to 30 m. Constant-depth girders deeper than 2.5 to 3.0 m are unusual and therefore for spans greater than 50 m consideration should be given to varying-depth girders through providing a curved soffit or haunches. For haunch lengths of 20 to 25% of the span from the pier, internal span-to-depth ratios of 18 at the pier and as little as 30 at midspan are normally used.

Single-cell box girders provide the most efficient section for casting – these days multicell boxes are rarely used in this method of construction. Inclined webs improve aesthetics but introduce added difficulties in formwork when used in combination with varying-depth girders. The area of

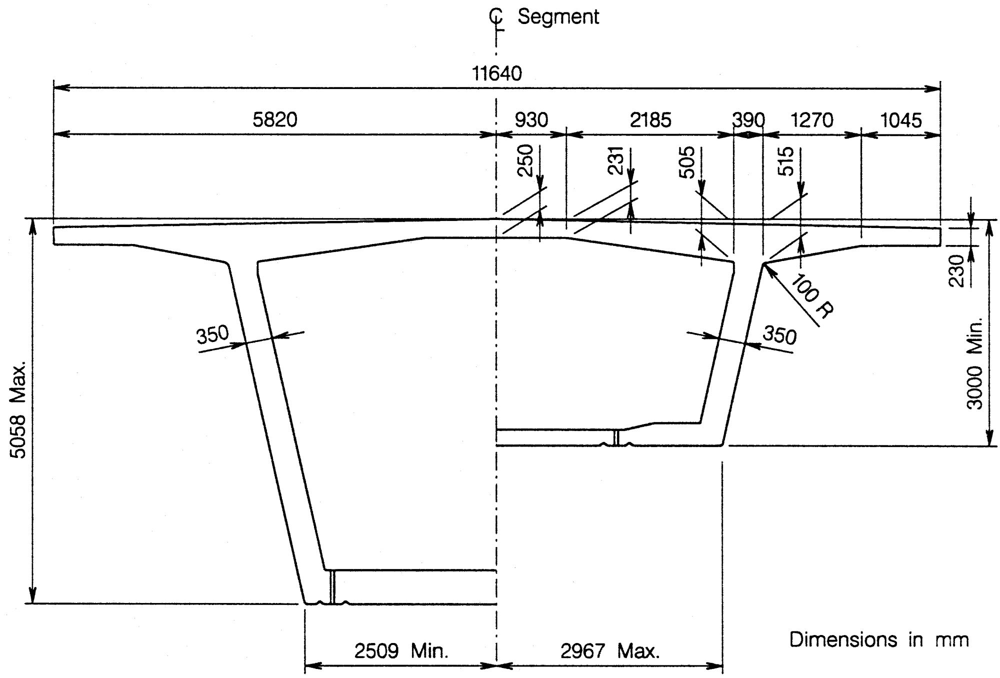


FIGURE 11.2 Typical cross section of a varying-depth girder for a 93-m span.

the bottom slab at the pier is determined by the modulus required to keep bottom fiber compressions below the allowable maximum at this location. In the case of internal tendons local haunches are used at the intersection of the bottom slab and the webs to provide sufficient space for accommodating the required number of tendon ducts at midspan. The distance between the webs at their intersection with the top slab is determined by achieving a reasonable balance between the moments at this node. Web thicknesses are determined largely by shear considerations with a minimum of 250 mm when no tendon ducts internal to the concrete are present and 300 mm in other cases. Figure 11.2 shows the typical dimensions of a varying-depth box girder.

11.2.3 Cast-in-Place Balanced Cantilever Bridges

The cast-in-place technique is preferred for long and irregular span lengths with few repetitions. Bridge structures with one long span and two to four smaller spans usually have a varying-depth girder to carry the longer span, hence making the investment in a mold which accommodates varying-depth segments even more uneconomical. A prime example of application of balanced cantilevering in an urban environment to avoid disruption to existing road services below is the structure of the Bangkok Light Rail Transit System, where it crosses the Rama IV Flyover (see Figure 11.3). The majority of the 26-km viaduct structure is precast, but at this intersection a 60-m span was required to negotiate the existing road at a third level with the flyover in service below. A three-span, 30-, 60-, 30-m structure was utilized with a box-girder depth of 3.5 m at the pier and 2.0 m at midspan and a parabolic curved soffit. The flyover was only disrupted a few nights during concrete placement of the segments directly above as a precaution.

In the above example, the side spans were constructed by balanced cantilevering; however, ideal arrangement of spans normally provides end spans which are greater than half the internal spans. These, therefore, cannot be completed by balanced cantilevering, and various techniques are used to reach the abutments. The most economical and common method is the use of falsework; however,

FIGURE 11.3 Construction of the Bangkok Transit System over Rama IV Flyover, Thailand.

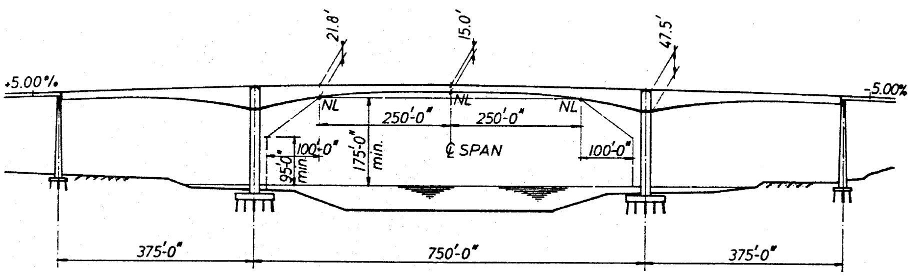


FIGURE 11.4 Houston Ship Channel Bridge, United States.

should the scale of the project justify use of an auxiliary truss to support the formwork during balanced cantilevering, then this could also be used for completing the end spans.

Another example of a cast-in-place balanced cantilever bridge is the Houston Ship Channel Bridge where a three-span, 114-, 229-, 114-m structure was used over the navigation channel (see Figure 11.4). A three-web box girder carrying four lanes of traffic is fixed to the main piers to make the structure a three-span rigid frame. Unusual span-to-depth ratios were dictated by the maximum allowable grade of the approach viaducts and the clearance required for the ship channel. The soffit was given a third-degree parabolic profile to increase the structural depth near the piers in order to compensate for the very limited height of the center portion of the main span. Maximum depth at the pier is 14.6 m, a span-to-depth ratio of 15.3 to enable a minimum depth at midspan of 4.6 m, and a span-to-depth ratio of 49. The box girder is post-tensioned in three dimensions: four 12.7-mm strands at 600-mm centers transversely in the top slab as well as longitudinal and vertical post-tensioning in the webs.

11.2.4 Precast Balanced Cantilever Bridges

Extending segmental construction to balanced cantilevering, and hence eliminating the need for falsework as well as substantial increases in the rate of construction, requires a huge leap in the technology of precasting: match casting. The very first bridge that benefited from match-casting technology was the Choisy-le-Roi Bridge near Paris, designed by Jean Muller and completed in 1964. This method has since grown in popularity and sophistication and is used throughout the world today. The essential feature of match casting is that successive segments are cast against the adjoining segment in the correct relative orientation with each other starting from the first segment away from the pier. The segments are subsequently erected on the pier in the same order, and hence no adjustments are necessary between segments during assembly. The joints are either left dry or made of a very thin layer of epoxy resin, which does not alter the match-cast geometry. Post-tensioning may proceed as early as practicable since there is no need for joints to cure.

The features of this method that provide significant advantages over the cast-in-place method, provided the initial investment in the required equipment is justified by the scale of the project, are immediately obvious and may be listed as follows:

- Casting the superstructure segments may be started at the beginning of the project and at the same time as the construction of the substructure. In fact, this is usually required since the speed of erection is much faster than production output of the casting yard and a stockpile of segments is necessary before erection begins.
- Rate of erection is usually 10 to 15 times the production achieved by the cast-in-place method. The time required for placing reinforcement and tendons and, most importantly, the waiting time for curing of the concrete is eliminated from the critical path.
- Segments are produced in an assembly-line factory environment, providing consistent rates of production and allowing superior quality control. The concrete of the segments is matured, and hence the effects of shrinkage and creep are minimized.

The success of this method relies heavily on accurate geometry control during match casting as the methods available for adjustments during erection offer small and uncertain results. The required levels of accuracy in surveying the segments match-cast against each other are higher than in other areas of civil engineering in order to assure acceptable tolerances at the tip of the cantilevers.

The size and weight of precast segments are limited by the capacity of transportation and placing equipment. For most applications segment weights of 40 to 80 tons are the norm, and segments above 250 tons are seldom economical. An exception to the above is the recent example of the main spans of the Confederation Bridge where complete 192.5-m-long balanced cantilevers weighing 7500 tons were lifted into place using specialized equipment (see Figure 11.5). The 250-m main spans of this fixed link in Atlantic Canada, connecting Cape Tormentine, New Brunswick, and Borden, Prince Edward Island, were constructed by a novel precasting method. The scale of the project was sufficiently large to justify precast segmental construction; however, adverse weather and site conditions provided grounds for constructing the balanced cantilevers, 14 m deep at the piers, in a similar method to cast-in-place construction but in a nearby casting yard. The completed balanced cantilevers were then positioned atop completed pier shafts in a single operation. A light template match-cast against the base of the pier segment allowed fast and accurate alignment control on the spans.

11.2.5 Loads on Substructure

The methods for supporting the nominal out-of-balance forces during balanced cantilevering were described earlier. The following forces should be considered in calculating the possible out-of-balance forces:

- In precast construction, one segment out of balance and the loss of a segment on the balancing cantilever as an ultimate condition;
- In precast construction, presence of a stressing platform (5 to 10 tons) on one cantilever only or the loss of the form traveler in the case of cast-in-place construction;

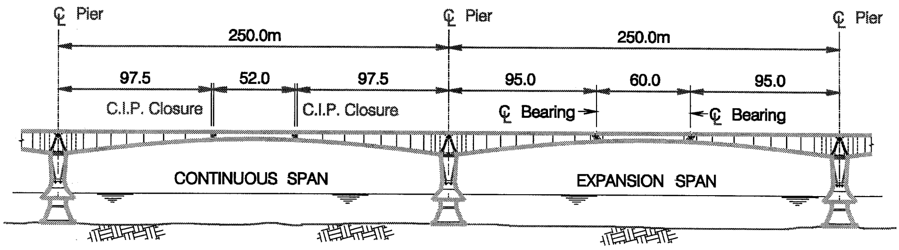


FIGURE 11.5 Main spans of the Confederation Bridge, Canada.

- Live loading on one side of 1.5 kN/m^2;
- Wind loading during construction;
- The possibility of one cantilever having a 2.5% higher dead weight than the other.

The loads on the substructure do not usually govern the design of these elements provided balanced cantilever construction is considered at the onset of the design stage. The out-of-balance forces may provide higher temporary longitudinal moments than for the completed structure; however, in the case of a piled foundation, this usually governs the arrangement and not the number of the piles.

11.2.6 Typical Post-Tensioning Layout

Post-tensioning tendons may be internal or external to the concrete section, but inside the box girder, housed in steel pipes, or both. External post-tensioning greatly simplifies the casting process and the reduced eccentricities available compared with internal tendons are normally compensated by lower frictional losses along the tendons and hence higher forces.

The choice of the size of the tendons must be made in relation to the dimensions of the box-girder elements. A minimum number of tendons would be required for the balanced cantilevering process, and these may be anchored on the face of the segments, on internal blisters, or a combination of both. After continuity of opposing cantilevers is achieved, the required number of midspan tendons may be installed across the closure joint and anchored on internal bottom blisters. Depending on the arrangement and length of the spans, economies may be made by arranging some of the tendons to cross two or more piers, deviating from the top at the piers to the bottom at midspan, thereby reducing the number of anchorages and stressing operations. External post-tensioning is best used for these continuity tendons which would allow longer tendon runs due to the reduced frictional losses. Where the tendons are external to the concrete elements, deviators at piers, quarterspan, and midspan are used to achieve the required profile. An example of a typical internal post-tensioning layout is shown in Figure 11.6.

11.2.7 Articulation and Hinges

The movements of the structure under the effects of cyclic temperature changes, creep, and shrinkage are traditionally accommodated by provision of halving joint-type hinges at the center of various spans. This practice is now discontinued due to the unacceptable creep deformations that occur at these locations. If such hinges are used, these are placed at contraflexure points to minimize the effects of long-term deflections. A development on simple halving joints is a moment-resisting joint, which allows longitudinal movements only. All types of permanent hinges that are more easily exposed to the elements of water and salt from the roadway provide maintenance difficulties and should be eliminated or reduced wherever possible.

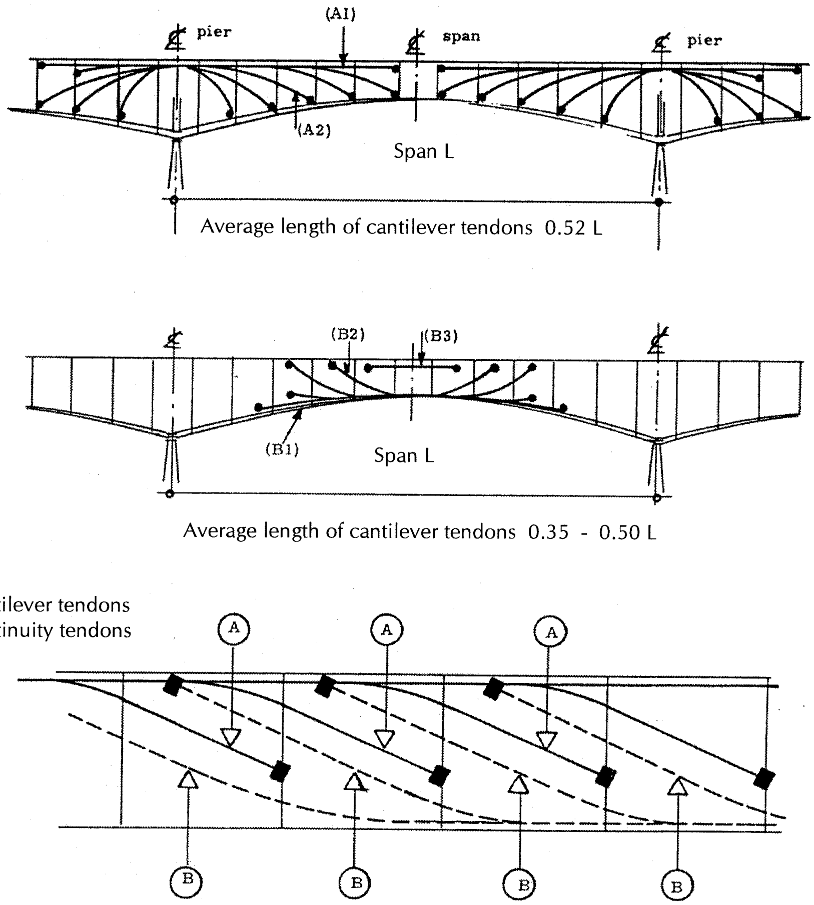


FIGURE 11.6 Typical post-tensioning layout for internal tendons.

If the piers are sufficiently flexible, then a fully continuous bridge may be realized with joints at abutments only. When seismic considerations are not a dominant design feature and a monolithic connection with the pier is not essential, bearings atop of the piers are preferred as they reduce maintenance and replacement cost. In addition, it will allow free longitudinal movements of the deck. A monolithic connection or a hinged bearing at one or more piers would provide a path for transmitting loads to suitable foundation locations.

11.3 Progressive and Span-by-Span Constructed Bridges

11.3.1 Overview

In progressive or span-by-span construction methods, construction starts at one end and proceeds continuously to the other end. Generally, progressive construction is used where access to the ground level is restricted either by physical constraints or by environmental concerns. Deck variable cross sections and span lengths up to 60 m are easily accommodated. In contrast, span-by-span precast segmental construction is used typically where speed of construction is of major concern. Span lengths up to 50 m are most economical as it minimizes the size of the erection equipment.

FIGURE 11.7 Fréburge Viaduct, France—erection with movable stay tower.

11.3.2 Progressive Construction

The progressive method step-by-step erection process is derived from cantilever construction, where segments are placed in a successive cantilever fashion. The method is valid for both precast and cast-in-place segments. Due to the excessively high bending moments the cantilever deck has to resist over the permanent pier during construction, either a temporary bent or a temporary movable tower–stay assembly would have to be used. As shown in Figure 11.7, for precast construction using a temporary tower and stay system, segments are transported over the erected portion of the bridge to the end of the completed portion. Using some type of lifting equipment, e.g., a swivel crane, the segment is placed in position and supported temporarily either by post-tensioning to the previous segment or by stays from a tower.

The advantages of this methods are

- Operations are conducted at deck level.
- Reactions on piers are vertical.
- The method can easily accommodate variable horizontal curves.

The disadvantages are

- The first span is erected on falsework.
- Forces in the superstructure during erection are different from those in the completed structure.
- The piers are temporarily subjected to higher reactions from dead load than in the final structure because of the length of the cantilever erected. However, considering the other loads in the final structure, this case is not generally controlling the pier design.

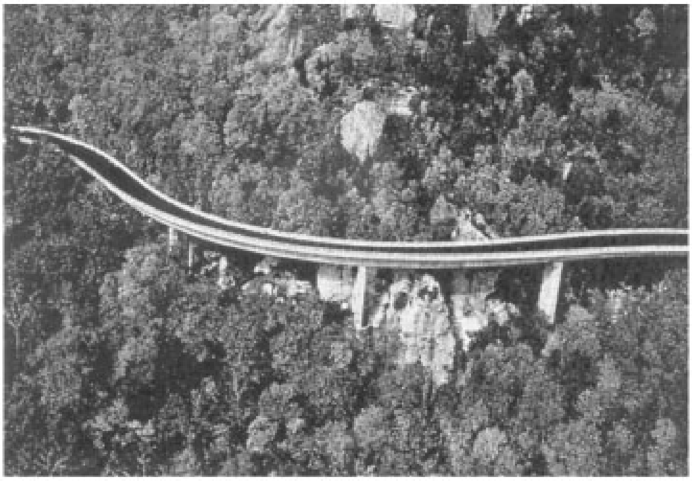


FIGURE 11.8 Completed Linn Cove Viaduct, United States.

FIGURE 11.9 Linn Cove Viaduct — pier being constructed from the deck level.

The Linn Cove Viaduct (1983) on the Blue Ridge Parkway in North Carolina shown in Figure 11.8, demonstrated the potential progressive placement when one is forced to overcome extreme environmental and physical constraints. Because access at the ground level was limited, the piers were constructed from the deck level, at the tip of an extended cantilever span. Temporary cable stays could not be used due to the extreme horizontal curvature in the bridge. Instead, temporary bent supports were erected between permanent piers. Figure 11.9 shows one temporary support in the background while a permanent precast pier is being erected from the deck level.

FIGURE 11.10 Lifting completed span of the Seven Mile Bridge, Florida, using an *overhead truss.*

11.3.3 Span-by-Span Construction

As with balanced cantilever and progressive placement, span-by-span construction activity is performed primarily at the deck level and typically implemented for long viaducts having numerous, but relatively short spans, e.g., <50 m. It was initially developed as a cast-in-place method of construction, on formwork, with construction joints at joint of contraflexure. The form traveler is supported either on the bridge piers, on the edge of the previously erected span and the next pier or, at times, even at the ground level. With the precast segmental method, segments are placed and adjusted on a steel erection girder spanning from pier to pier, then post-tensioned together in one operation. Although both the cast-in-place and the precast span-by-span construction methods continue to be used, precast segmental has become the method of choice for most applications.

Long Key and Seven Mile Bridges, United States: Two early applications of the precast span-by-span method are the Long Key Bridge (1977) and the Seven Mile Bridge (1978), both located in the Florida Keys. The shorter, 3000-m, 100-span Long Key Bridge is the first application of precast span-by-span construction with dry segment joints and external post-tensioning in the United States.

Essentially the same bridge design concept as Long Key — only much longer — the 10,931-m, 270-span Seven Mile Bridge utilized rectangular precast piers and an overhead truss, as shown in Figure 11.10. The overhead truss allowed easier repositioning from one span to the next one and thus improved overall erection speed.

Bang Na–Bang Pli–Bang Pakong Expressway, Thailand: A number of span-by-span highway and rail mega projects have been either completed recently or currently are under construction in Southeast Asia. Probably, the most innovative of these recent applications is the 54,000-m, 1300-span, Bang Na–Bang Pli–Bang Pakong Expressway. The girder supports segment assembly and span installation activities. This erection process can be regarded as "assembly-line" in that there is no requirement for disassembly and reassembly of the erection girder as it travels from pier to pier. The piers, although designed structurally for the construction process, can also be seen to provide an aesthetically pleasing, somewhat "floating," appearance to the six-lane, 27-m-wide box girder. Figure 11.11 shows one of the erection girders as it lifts a segment. With five erection girders erecting a span every 2 days or 780 m of superstructure per week, construction of the viaduct is expected to last approximately 2 years and be completed in 1999, without interruption of traffic below.

Roize, France: Another innovative example of span-by-span construction is the 112-m, three-span, prestressed composite truss Roize Bridge (1991) in the French Alps, shown in Figure 11.12. The deck is made of prestressed concrete and steel. Each factory-built tetrahedron module and

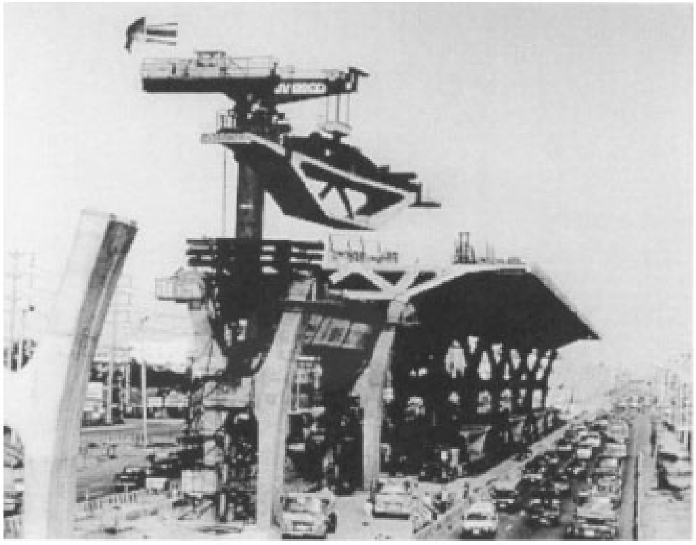


FIGURE 11.11 Bang Na Expressway, Thailand — launching of girder erection.

FIGURE 11.12 View of the Roize Bridge, France — space truss spans using tetrahedron modules.

precast pretensioned slab is placed on erection beams and adjusted into position. After welding the bottom member joints and casting the closure strips, the modules are post-tensioned together as a completed span. Due to the modular basis, this two-lane bridge represents a new class of super-lightweight, factory-built segments.

Channel Bridge, United States: The first precast, prestressed channel bridge in the United States was built in 1974 in San Diego, California, as a pedestrian crossing at San Diego State University. This concept was reused 18 years later as an experimental study for new bridge standards, initially

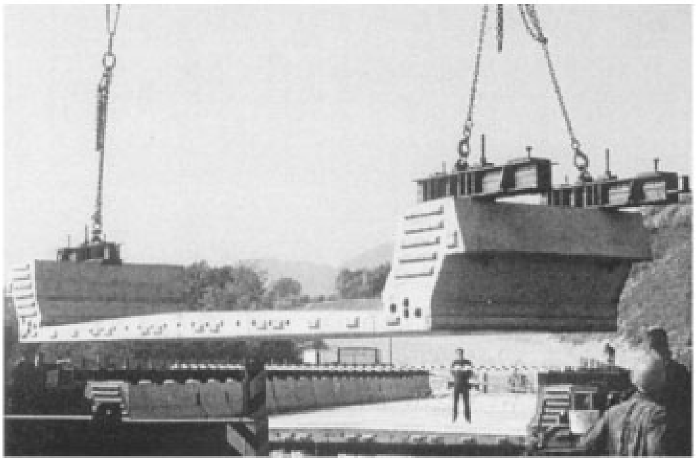


FIGURE 11.13 Channel Bridge, France, under construction.

by the French Highway Administration. Figure 11.13 shows the 54-m, two-span Champfeuillet Bridge (1992), under construction along the Rhône Alpine Motorway near Grenoble, France. The most innovative aspect of the concept is the use of the concrete parapets as part of the structure. With the primary longitudinal post-tensioning passing through the barriers, an extremely lightweight, shallow section is possible.

Research and implementation of the Channel Bridge, although continuing in Europe, also has begun recently in the United States. Initiated by the Federal Highway Administration (FHWA) and the Highway Innovative Technology Evaluation Center (HITEC), a branch of the Civil Engineering Research Foundation (CERF), at least two applications of the Channel Bridge concept have been completed recently in the United States for the New York State Department of Transportation (NYSDOT).

The primary benefits of the concept are as follows:

- Lightweight, easily placed segments.
- Fast erection times with small investment in erection equipment.
- Increased vertical clearance beneath the superstructure, because the load-carrying members are above the roadway slab, not below.
- A reduction in the number of bridge overpass piers required, which increases safety levels for traffic lanes below.

Span by span, as used today, utilizes post-tensioning tendons outside the concrete, but inside the box girder for ease of precasting and speed of installation together with dry joints, no epoxy, between segments. The post-tensioning tendons are continuous from pier segment to pier segment.

11.4 Incrementally Launched Bridges

11.4.1 Overview

The incremental launching technique has been used on bridges numbering in the hundreds since its introduction by Professor Fritz Leonhardt in 1961 for the Río Caroni Bridge in Venezuela. It is an effective alternative for the bridge designer to consider when the site meets its particular alignment requirements. The method entails casting the superstructure, or a portion thereof, at a stationary location behind one of the abutments. The completed or partially completed structure is then jacked into place horizontally, i.e., pushed along the bridge alignment. Subsequent segments can then be cast onto the already completed portion and in turn pushed onto the piers. Because all of the casting operations are concentrated at a location easily accessible from the ground, concrete quality of the same level expected from a precasting yard can be achieved. The procedure has the advantage that, like the balanced cantilever technique, it obviates the need for falsework to cast the girder. Moreover, heavy erection equipment, cranes, gantries, and the like, are not necessary, nor is the use of epoxy at segment joints. Usually, the only special equipment required is light steel truss work for a launching nose to reduce the cantilever moments during launching.

11.4.2 Special Requirements

There are two peculiarities associated with the technique, which must be appreciated by the designer. The first is that the alignment must be straight or, if it involves curves, the curvature must be constant. The second is that during launching, every section of the girder will be subjected to both the maximum and minimum moments of the span; and the leading cantilever portion will be subjected to slightly higher moments. This second constraint usually leads to slightly deeper sections, on the order of $\frac{1}{15}$ the span, than would otherwise be considered. The girders must also be of constant depth as each section will at sometime be supported on the temporary bearings. Other considerations include the necessity for a large area behind the abutment for the casting operations, the requirement to lift the bridge off of the temporary bearings, and place it on the permanent ones when launching is complete and the need for very careful control of geometry during casting.

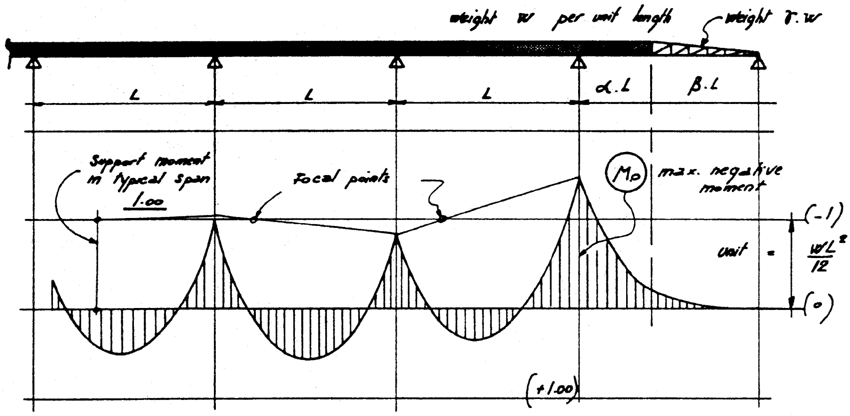
Incremental launching is generally considered for long viaducts with many spans of the same length. Spans up to 100 m can be considered; the requirement for constant-depth girders makes longer spans uneconomical. A single long span in the center of a project can be achieved by launching from both abutments and finishing at the long span with two converging cantilevers. The practical length limit for launching in about 1000 m. Bridges of twice this length can be considered by launching from both abutments.

11.4.3 Typical Post-Tensioning Layout

During superstructure launching each section of the girder is subjected to constantly reversing bending moments as it proceeds from temporary support to midspan. Because of the sign change in the applied moments, the efficient use of draped tendons for launching load effects is impossible. The general procedure has therefore been to apply axial prestressing for the launching operation. These tendons are usually straight, being contained in the top and bottom slabs of the girder. The tendons for successive segments must be spliced to these with couplers or stressed in buttresses in an overlapping fashion. This prestressing is subsequently augmented with either draped tendons or short top- and bottom-slab tendons for respective negative and positive moment regions in the completed structure to meet service state requirements. In some instances, permanent draped prestressing has been placed in the configuration required for the final condition, and temporary tendons with an opposing drape are provided to counteract their bending effects during launching. These temporary tendons are then removed when launching is complete.

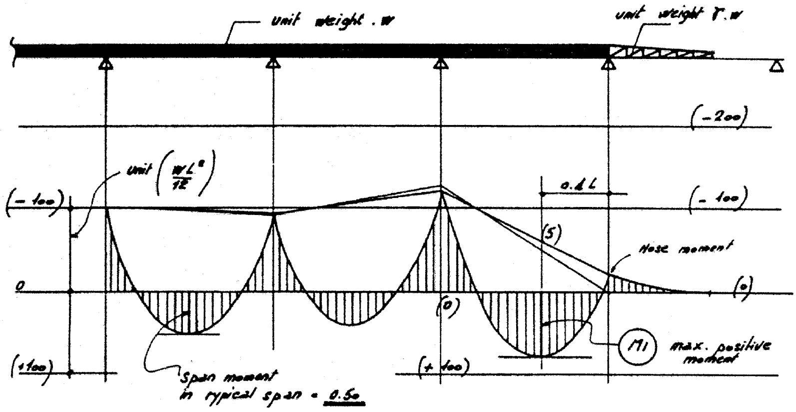
11.4.4 Techniques for Reducing Launching Moments

As suggested above, the launching moments in the leading spans, especially the first cantilever span, will be greater than those in the following interior spans. If the girder is simply launched to the first pier with no special provision to reduce these moments, they will in fact be on the order of six times the typical negative moment over a pier. The method used most frequently to overcome this problem has been a light structural-steel launching nose attached to the leading cantilever (see Figures 11.14 and 11.15). This nose supports the girder without the weight penalty of the heavier concrete section. In order to be effective, the nose must be both as light and as stiff as possible.



α	β	M_0
0.20	0.80	0.82
0.30	0.70	1.09
0.40	0.60	1.46
0.50	0.50	1.95
1.00	0.00	6.00

FIGURE 11.14 Critical negative moment during launching with nose. $M_1 = [WL^2/12)(6\alpha^3 + 6y)(1 - \alpha^3)$. Multiplier $= WL^2/12$. For $y = 0.11$.



α	β	M_1
0.20	0.80	0.74
0.30	0.70	0.79
0.40	0.60	0.83
0.50	0.50	0.86
1.00	0.00	0.93

FIGURE 11.15 Critical positive moment during launching with nose. $M_1 = [(WL^2/12)(0.933 - 2.96y\beta^2)$. Multiplier $= WL^2/12$. For $y = 0.11$.

For longer spans, the steel nose is not as effective, and other methods have been employed to reduce launching moments. Temporary piers are a viable solution when ground conditions are such that the foundation costs are relatively modest and the pier height is not too great. If either of these conditions is not found, the cost can escalate rapidly as a temporary pier will be required in every span.

One last method that has been employed successfully is a temporary pylon attached to the deck at the trailing end of the first span which supports stays connected to the leading end. This device is very efficient in reducing the cantilever moment in the leading span; however, it produces an undesirable positive moment when the pylon is at midspan. For this reason, the stays must be equipped with a jack to adjust the stay force as needed during the various stages of the launching operations.

11.4.5 Casting Bed and Launching Methods

Segment lengths for incrementally launched bridges are generally greater than for other types of segmental bridges. Typical segment lengths range from 15 to 40 m. Usually, a casting area twice the length of the segment is required for actual casting and the ancillary operations that must be conducted there. The casting bed is generally a significant structure itself, as the strict geometry-control requirements of the technique make settlement of the formwork unacceptable.

Launching has been accomplished in the past either by tendons attached to the girder and horizontal jacks bearing on the abutment or by a horizontal jack bearing on the abutment face connected to a vertical jack which slides on a bearing. The upper surface of the vertical jack is fitted with a friction device to bear on the soffit of the box girder. The vertical jack is inflated to provide the normal force required for transferring the launching force by friction.

11.5 Arches, Rigid Frames, and Truss Bridges

11.5.1 Arch Bridges

The first step toward the segmental construction of arches was taken shortly after World War I by Eugéne Freyssinet. He employed hydraulic jacks to lift the completed Villeneuve arch from its falsework by applying an internal thrust at its crown. This departure from the classical method of striking the centering to develop the thrust in the arch opened the door to modern arch construction techniques that do not rely on falsework. It also presented the opportunity to reduce the bending moments in the arch by eliminating the dead load bending associated with axial shortening of the ribs.

11.5.1.1 Arches Erected without Falsework

The development of stay-cable and form-traveler technology has made possible the erection of arches in cantilever fashion without a centering supported from below. One early example of this technique was the suite of viaducts built in Caracas, Venezuela, in 1952 (see Figure 11.16). The first quarter of the arch span was supported by light forms which were in turn supported by stay cables attached to a pilaster at the springing of the arch. The crown portion of the arch was then completed with a light centering supported on the already-completed portion of the arch so that no falsework was required in the valley below.

Several variations on this theme were subsequently developed. The methods employed varied, depending on site conditions, from the use of very high pylons with a single group of stays allowing construction of the arch all the way to the crown to those which used the permanent spandrel columns in conjunction with temporary stay diagonals to form a truss. These methods are summarized in Figure 11.17.

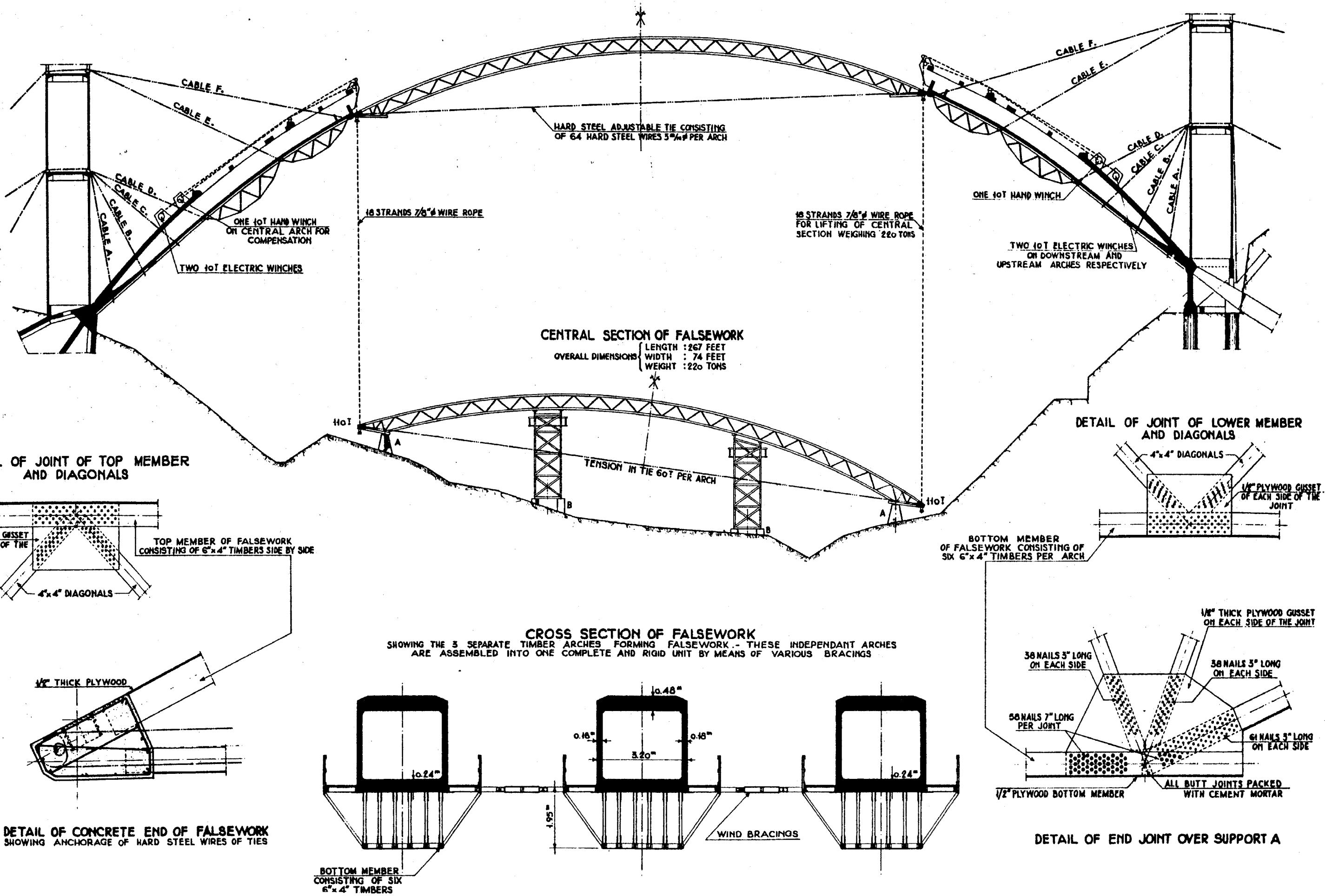


FIGURE 11.16 Caracas viaducts-erection of center portion of arch falsework.

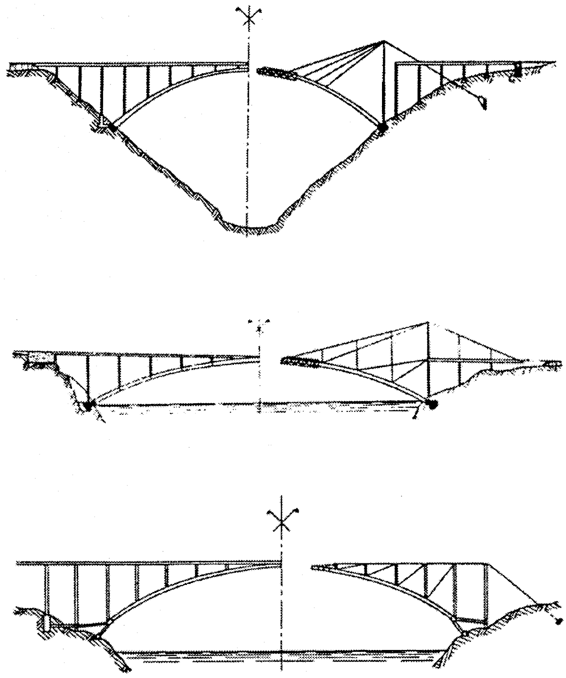


FIGURE 11.17 Various cantilevers—erection techniques for arches.

11.5.1.2 Precast Arches

The first precast segmental arch bridge was built in France in 1948. This bridge at Luzancy over the Marne River is composed of three box-section arches built up from 2.44-m-long precast segments. The finished span length is 55 m. Because of the severe clearance requirements, the arch has a very unusual span-to-rise ratio of 23 to 1. The segments were connected via 20-mm dry-packed joints and prestressing on the approach behind an abutment, with the resulting rib being moved to its final position by an aerial cableway.

Construction of concrete arches without falsework was employed almost exclusively in conjunction with the cast-in-place cantilever technique until the construction of the Natches Trace Bridge in Tennessee in 1993. This precast arch, which originally was designed for erection on a moveable falsework, was the first precast arch to be erected on stays. The unusual design, which omits spandrel columns, results in a slender appearance. There are two arches: one with a span of 177 m and a rise of 44 m, the other with a span of 141 m and a rise of 31 m. The arch segments are 4.9 m wide and vary in depth from 4 m at the springing to 3 m at the crown (see Figure 11.18).

11.5.2 Rigid Frames

Frame bridges can be considered a hybrid of arch and girder forms. They are an appropriate alternative to either of those types for intermediate span lengths. Rigid frame bridges are well suited to segmental construction techniques.

Rigid frame bridges often have some of the same site requirements as arch bridges. They are well suited to valleys and generally will require foundations capable of resisting large horizontal actions. Generally, some form of temporary support will be required until the frame is complete, meaning that construction techniques that eliminate falsework may need slight modification for these structures. One of the most aesthetically convincing applications of the rigid frame is the Bonhomme Bridge in Brittany, France (see Figure 11.19). This slant-leg frame was built using the cast-in-place balanced cantilever technique. Temporary piers were installed below the slant legs to support them

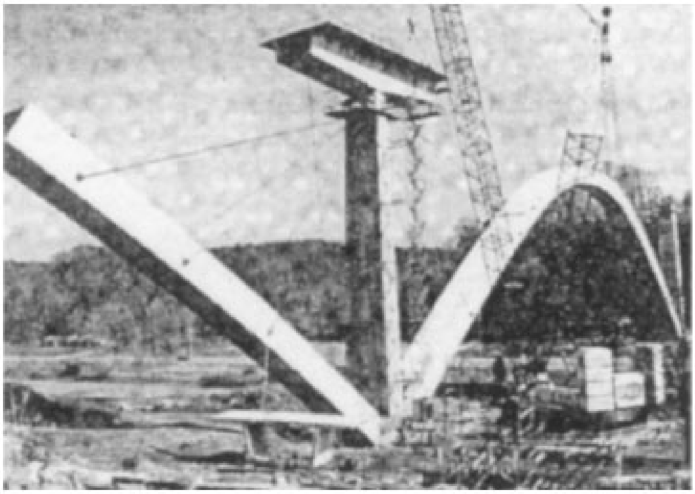


FIGURE 11.18 Natches truss arch—cantilever erection of ribs.

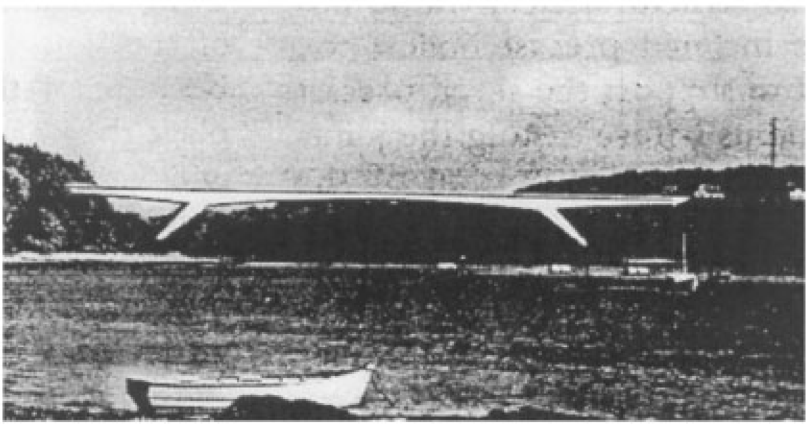


FIGURE 11.19 Bonhomme Bridge in Brittany, France.

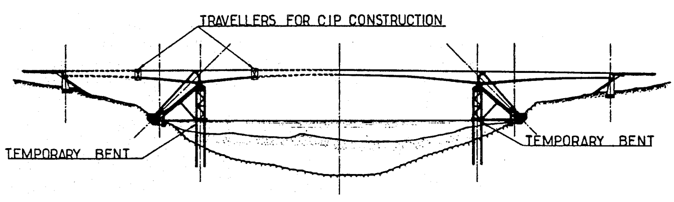


FIGURE 11.20 Temporary support for the Bonhomme Bridge.

before the thrust was developed in the frame (see Figure 11.20). Jacks under the temporary-support piers and at the midspan closure were used to adjust the geometry before closing the span.

11.5.3 Segmental Trusses

Although relatively few examples have been built, segmental trusses are interesting, especially for long spans, in that they offer very efficient use of materials. This economy translates directly into lighter elements and smaller loads to be dealt with during construction, as well as reduced material cost.

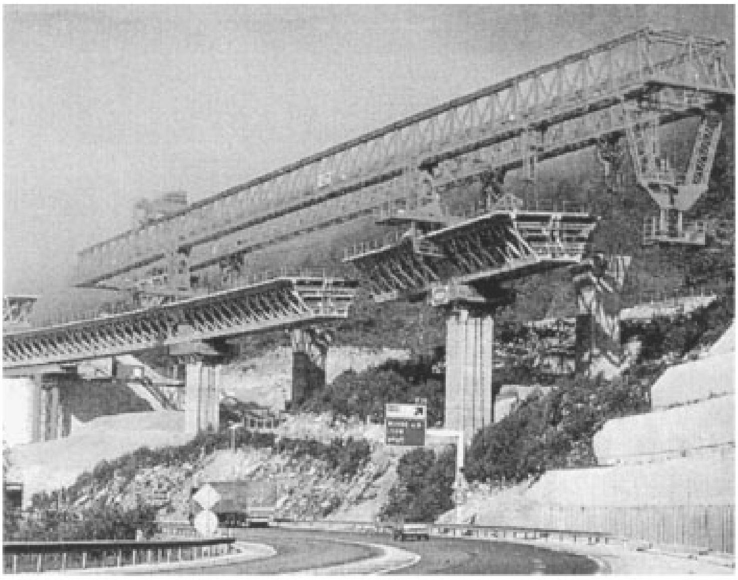


FIGURE 11.21 Cantilever erection of the Viaduct des Glacièrs, France.

One of the earliest segmental trusses was the Mangfallbrücke in Austria, which was constructed in 1959. It had a total length of 288 m with a maximum span of 108 m and was constructed by the cast-in-place segmental technique in conjunction with temporary piers.

Later examples were developed in precast segmental, of which the Viaduc de Sylans and the Viaduc des Glacieres are the most notable. These sister structures were constructed in balanced cantilever with a self-launching overhead truss (see Figure 11.21). The segments were prestressed in three directions with a combination of external and internal tendons. "X" members for the open webs were precast and subsequently placed in the molds prior to segment casting.

The most recent development in segmental trusses is the composite truss. This concept employs concrete for the top and bottom chords and steel sections for the open webs. In some cases, however, steel is used for the tension chord as well. An excellent example of this type of construction is the bridge over the Roize in France, built with the span-by-span method. This structure was conceived as a truss work of factory-produced steel truss work and precast slabs. These two elements were joined at the site by cast-in-place joints and external tendons (see Figure 11.22). The precast slabs served as the top (compression) chord while a hexagonal steel tube served as the bottom chord. The resulting structure is equally viable as the deck for short-span viaducts and stiffening girder for long-span cable-supported bridges.

11.6 Segmental Cable-Stayed Bridges

11.6.1 Overview

Theories on cable-stayed bridges are presented in another chapter. We shall address here cable-stayed bridges only as they relate to segmental construction. In the majority of segmental cable-stayed bridges, the methods of construction fall in the three following categories, by order of importance:

- Cantilever construction
- In-stage construction
- Push-out construction

FIGURE 11.22 The Roize Bridge, France — erection of steel bottom chord and webs.

The choice of material depends upon many factors and load conditions; it should be remembered that concrete is an excellent material for cable-stayed structures, because of its properties in resisting compression and its mass and damping characteristics in resisting aerodynamic vibrations. For the proposed Ceremonial Bridge in Malaysia, with a main span of 1000 m and a single plane of stays, concrete deck in the pylon area is associated with a composite cross section toward the center of the span. Comparative studies show that the replacement of the composite section with its concrete slab by an orthotropic slab would adversely affect the project because of its lack of mass.

11.6.2 Cantilever Construction

11.6.2.1 Design

It is important to keep the project simple and pay attention to details to achieve economy and efficiency during construction.

The length of segments must be equal and, depending upon the spacing of stays, the segment joints must be such that a stay always falls in the same location within a segment. If the segments are long, the stay should be located toward the free end of the segment. Cross sections must be kept constant as much as possible, the variations being limited to the web and bottom slab thickness. The post-tensioning layout must be repetitive from segment to segment (see Figure 11.23). Erection phases are critical in terms of stability and stresses. Wind effects on the partially built structure must be investigated for static and dynamic effects. A shorter return period is usually used during construction (10 years). Seismic effects must also be investigated in areas prone to earthquakes. To increase stability, temporary cables can be installed at a certain stage of completion.

Stresses in the main elements of the structure often reach a maximum during construction, and the final state of stresses in the finished structure depends greatly on the accuracy of construction. Figure 11.24 shows a typical erection cycle. It is important that all erection phases be reviewed to ensure that the stresses are within allowable limits at each stage.

Stay forces are large and applied on very localized areas of the deck, and their local effects must be analyzed in detail. For instance, the stays apply high, concentrated forces on the section, at the middle, in the case of a single plane of stays, or at the edges with two planes of stays. These forces are not immediately available in the whole cross section, but are spread out at approximately 45°. This shear lag effect is more critical during construction than in service. Construction phases should be checked, assuming a 45° distribution of the horizontal component of the stay force while the vertical component is effectively applied at the stay anchorage (a finite-element computer program will generate the exact cross section stress distribution).

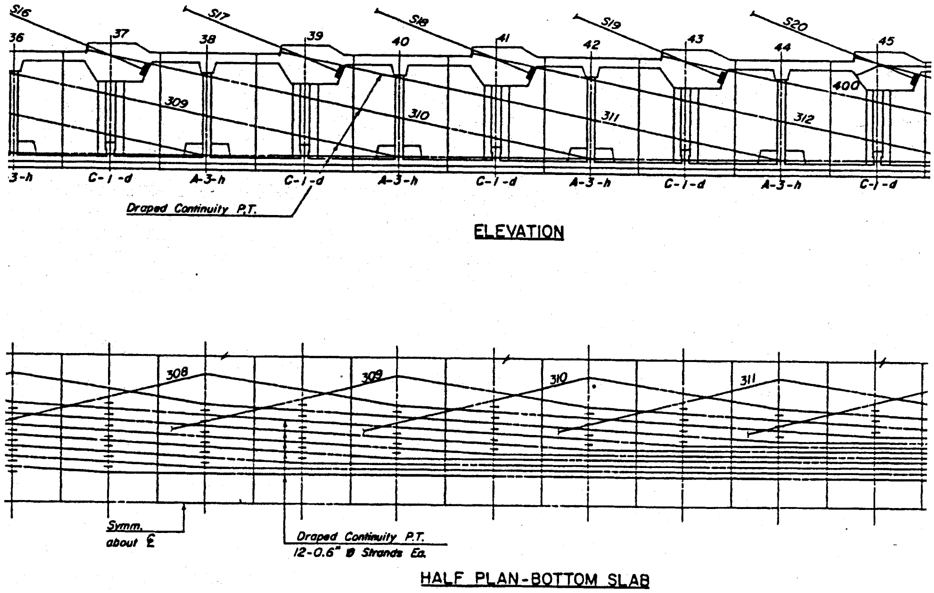


FIGURE 11.23 Sunshine Skyway, Florida — stay cables and post-tensioning layout.

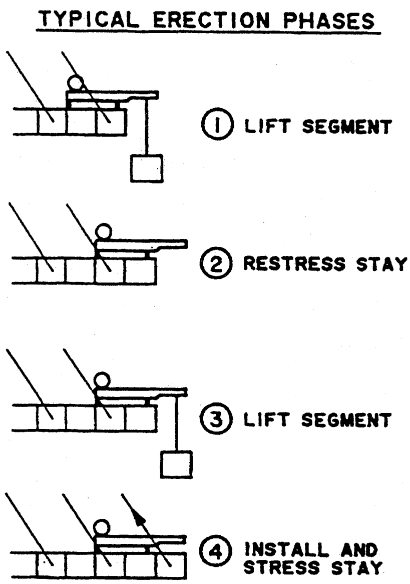


FIGURE 11.24 Typical erection phases.

This analysis usually shows the necessity of adding a temporary post-tensioning system toward the end of the cantilever, in the area outside the stay centerline (see Figure 11.25).

When a stay is anchored in an already constructed deck, such as a backstay anchored in the side span, the horizontal component of the stay force is distributed half in compression in front of the anchor and half in tension in the back of the anchor. This is called the entrainment effect; care must be taken to have enough tension capacity behind the anchor, either rebars or available compression, to prevent cracking or opening of the joint in case of precast construction.

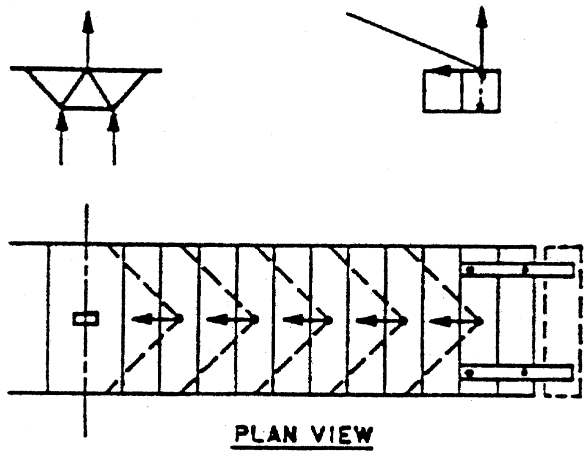


FIGURE 11.25 Shear lag during construction.

11.6.2.2 Cantilever Cast-in-Place Construction

Cast-in-place stayed bridges are built according to the same general principle as a typical box-girder bridge. After the pylon has been built up to the first pylon stay anchorage points and the starting deck segment at the pylon cast, travelers can be installed and cantilever construction started. Temporary stays are sometimes necessary to carry the weight of the traveler plus the newly cast segment before the permanent stay pertaining to that segment can be installed and stressed especially for thin, small inertia decks. A more elegant way is to use the permanent stay, which can be anchored in a precast anchorage block secured to the traveler. The horizontal component of the stay force is carried either by the traveler (see Figure 11.26) or by a precast member, which becomes part of the future segment. The permanent stay can also be anchored in the final deck if the stay anchor structure is staggered ahead of the whole section. This was the case at the Isère Bridge shown in Figures 11.27 and 11.28, with the center spine where the stays are anchored was cast in a first phase and the remainder of the section in a second phase. The phases are as follows:

- Launching of traveler;
- Concreting of the center spine (8 m) and stressing stay to 35% of its final force;
- Launching side forms, connecting bottom slab, and stressing stay to 70% of its final force;
- Concreting top slab and stressing stay to 80% of its final force.

11.6.2.3 Cantilever Precast Construction

Precast segmental bridges become economically feasible for relatively large bridges where the cost associated with setting up a casting yard can be offset by the speed of casting segments and the speed of erection. It is very interesting if the approaches to the main span are also precast segmentally, because then the cost of equipment is written off on an even larger volume.

A great example is the Sunshine Skyway Bridge in Florida, with a main span of 366 m for a total length of 1220 m, where the same cross section is used throughout the high-level bridge (see Figures 11.29 and 11.30). The 120-ton segments were precast in a yard close to the site and delivered by barge. They were lifted into place by beam-and-winch assemblies mounted on the previously completed portion of the deck. The same lifting equipment was used for the high approaches to the main spans. The low-level approaches were made of two parallel box girders.

For the James River Bridge in Virginia, the same twin parallel precast box girders were used from one end of the bridge to the other. For the main span, a single plan of stays was used and the two boxes were connected by a transverse frame at each stay anchor location (see Figures 11.31 and

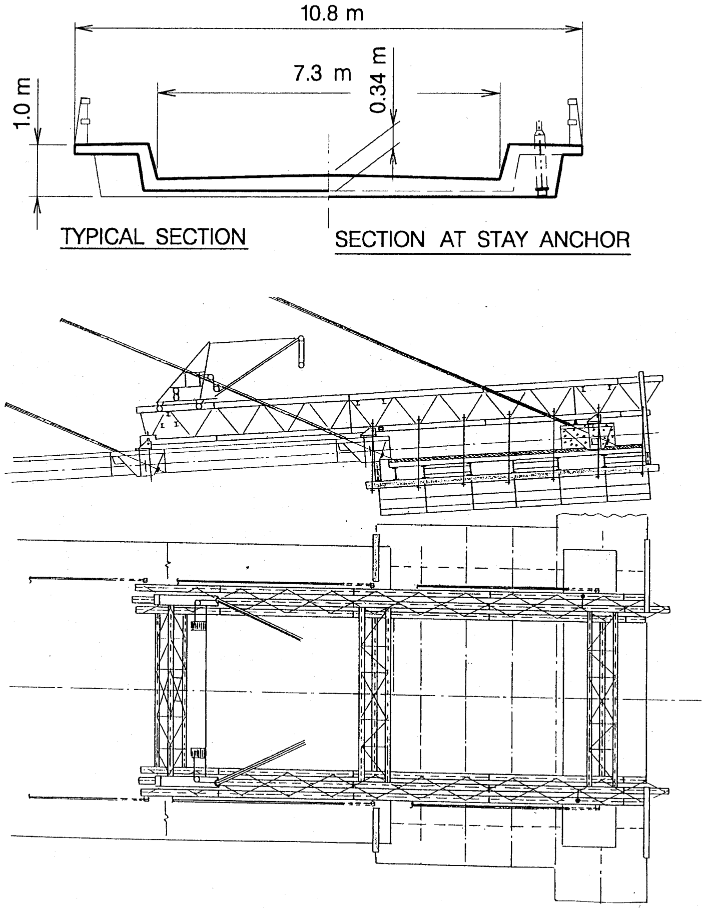


FIGURE 11.26 Santa Rosa Bridge, Bolivia – general view, cross section, and elevation.

11.32). With this scheme, construction can be carried out at deck level, with the segments erected by the span-by-span method. The main span can be built with crane-type lifting equipment mounted on the completed portion of the deck or, if desired, with cranes at ground level or on barges in the river.

11.6.2.4 Structural Steel Segmental Cantilever Construction

Cantilever construction can be applied to steel structures as well, the most recent example being the Normandie Bridge with an 856-m main span and 43.5-m approach spans. Concrete box girders are used for the approaches and part of the main span. The approaches were constructed by incremental launching and the first 116 m out from the pylon by segmental cast-in-place balanced cantilever techniques. Steel segmental construction is used for the remaining 640 m of the main span because of its light weight. The 19.65-m steel segments are barged to the site, lifted in place, secured against the previous segment, and then welded (see Figure 11.33).

11.6.3 In-Stage Construction

With this method, the deck is cast on a fixed soffit, with the side forms moving as the segments are cast. Stays can be installed during the casting, then stressed afterward. The advantage is that the bridge does not go through high-stress-level stages during erection and is practically built in its final stage. This method is only a variation of the cast-in-place scheme.

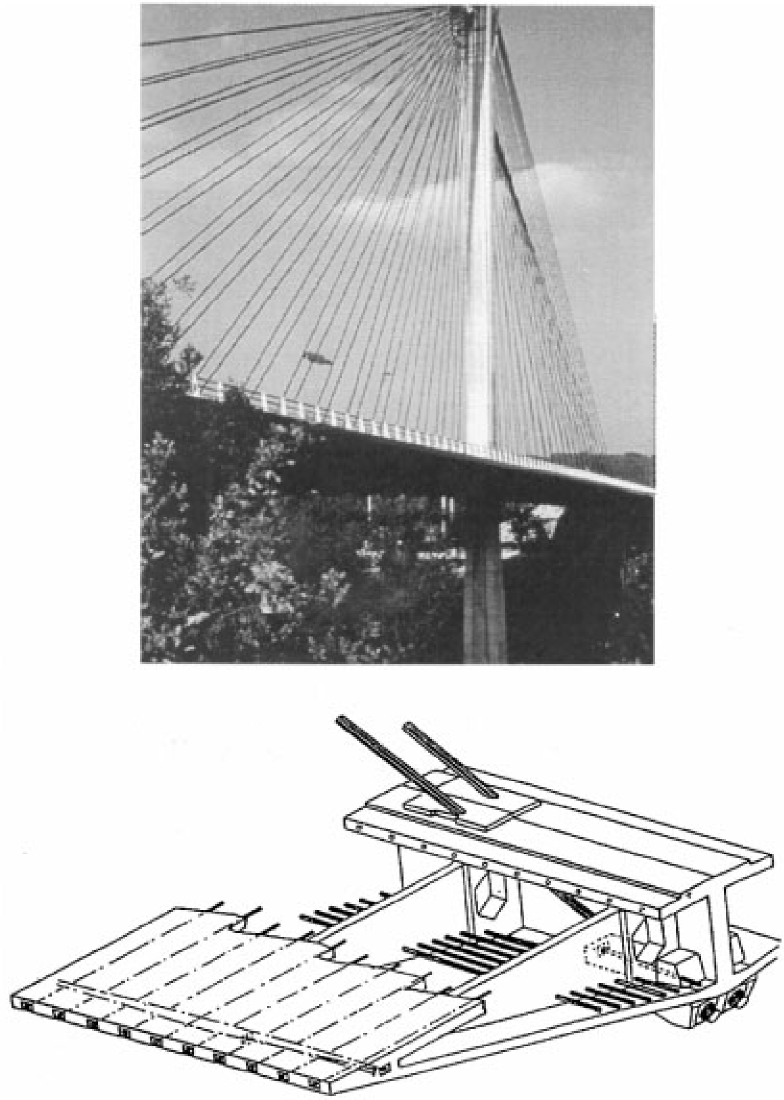


FIGURE 11.27 Isère Bridge, France — general and isometric views.

11.6.4 Push-Out Construction

This method is rarely used and not well adapted to cable-stayed bridges. Its use is restricted to sites where temporary supports can be installed. During pushing, the deck is subjected to large moment variations so steel decks are more suitable.

11.7 Design Considerations

11.7.1 Overview

The intent of this section is to present conditions that the designer should be aware of to produce a satisfactory design. The segmental technique is closely related to the method of construction and the structural system employed. It is usually identified with cantilever construction, but special attention must also be exercised with other methods, such as span-by-span, incremental launching, or progressive placement.

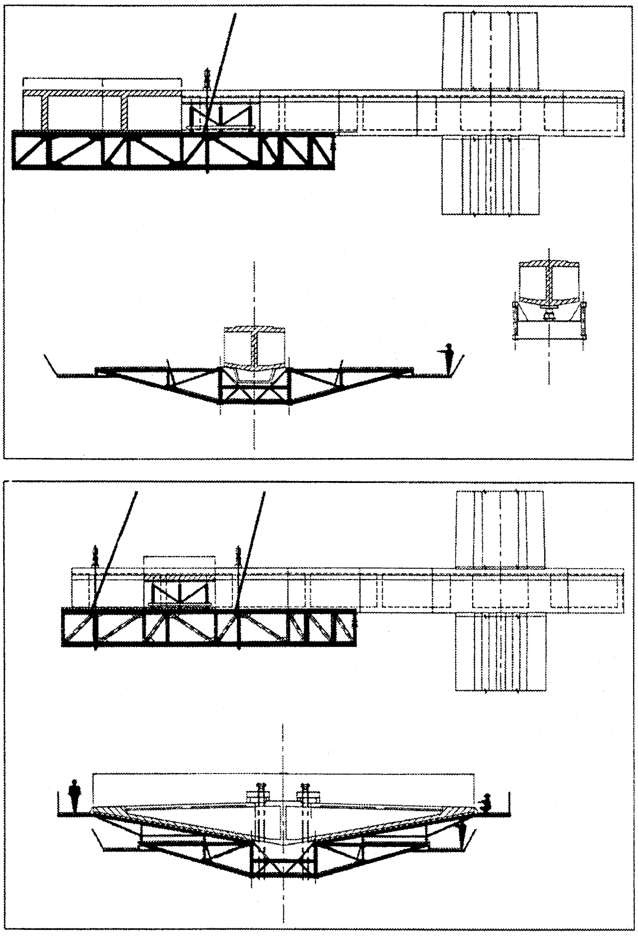


FIGURE 11.28 Isére Bridge — casting sequence.

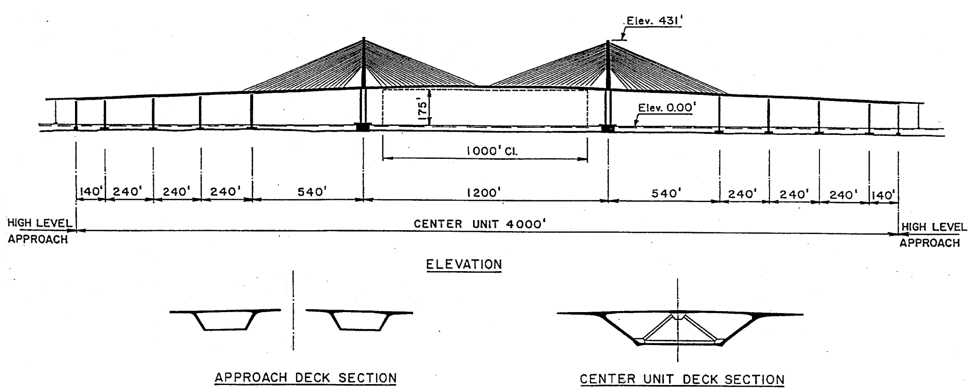


FIGURE 11.29 Sunshine Skyway Bridge, Florida — elevation.

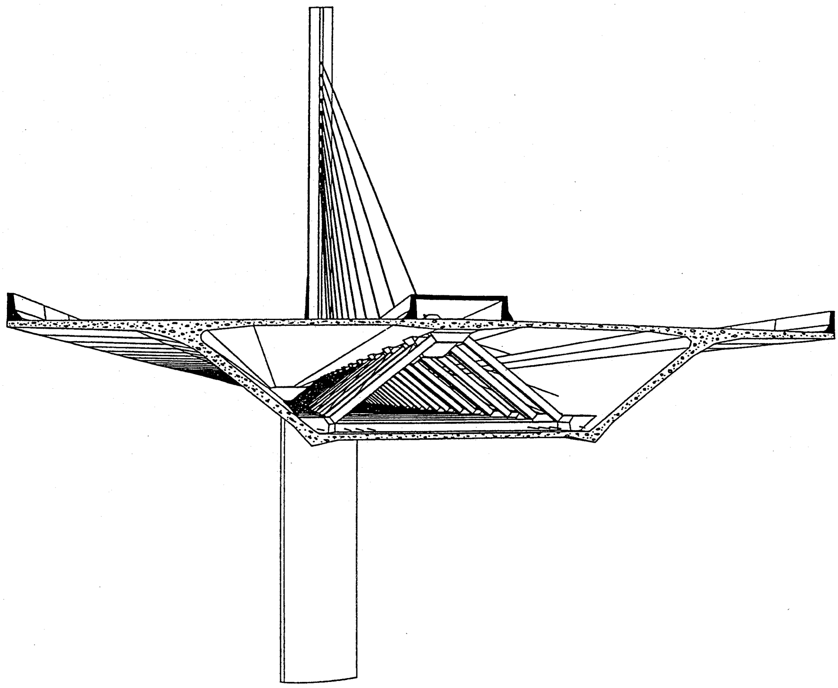


FIGURE 11.30 Sunshine Skyway Bridge — isometric view.

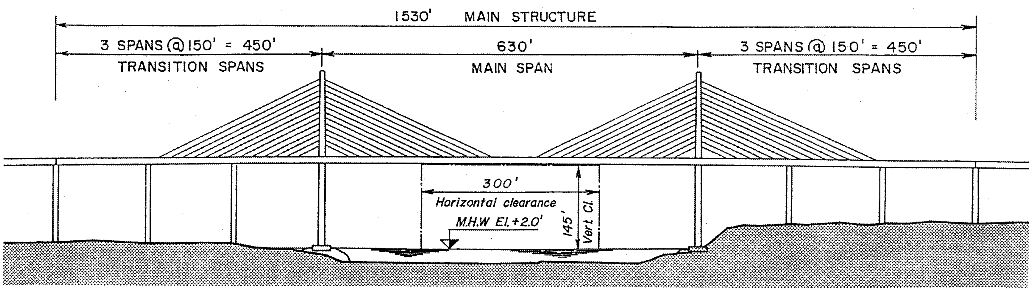


FIGURE 11.31 James River Bridge, Massachusetts – elevation.

11.7.2 Span Arrangement

11.7.1.1 Balanced Cantilever Construction

The span arrangement should avoid spans of significantly different lengths, if possible. This takes best advantage of the construction method by using cantilevers which are balanced about the column. The abutment spans of bridges built with this method are typically 60 to 65% of the central span length. These shorter end spans minimize the length of the bridge adjacent to the abutment, which must be built by using a different method, typically one employing falsework. Spans shorter than this may require a detail to resist uplift at the abutment resulting in live loading on the adjacent span (see Figure 11.34).

11.7.1.2 Span-by-Span Construction

For span-by-span construction the averaging of adjacent span lengths is not required, although it is advantageous to maintain similar span lengths adjacent to one another. The length of the abutment

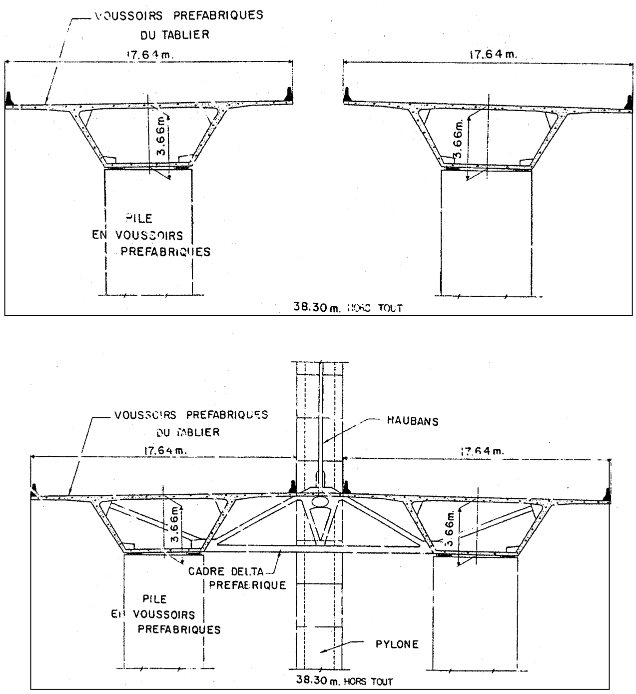


FIGURE 11.32 James River Bridge — cross sections.

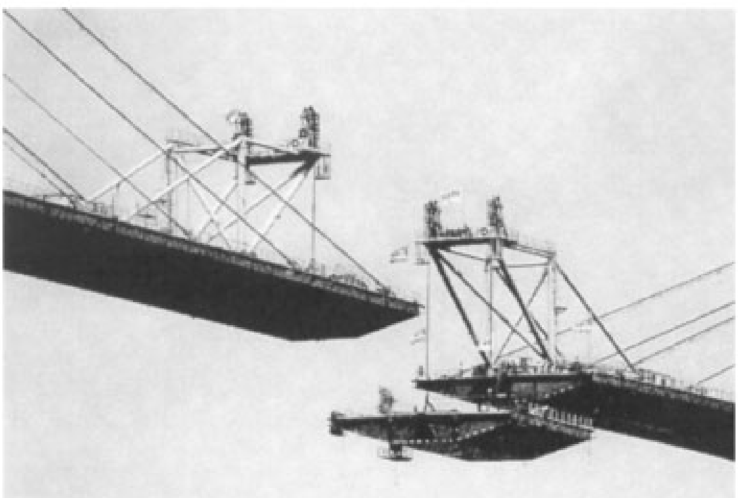


FIGURE 11.33 Normandie Bridge, France — lifting of a a steel segment.

or end span is typically kept the same as the interior spans. This is reasonable for this type of construction, since the secondary moments due to post-tensioning in the end spans are less than for the interior spans, and the post-tensioning requirement is therefore similar.

11.7.1.3 Location of Expansion Joints

Concrete bridge decks have been built with a length up to 1220 m between expansion joints and have had acceptable performance. The placement of expansion joints within a longer viaduct may

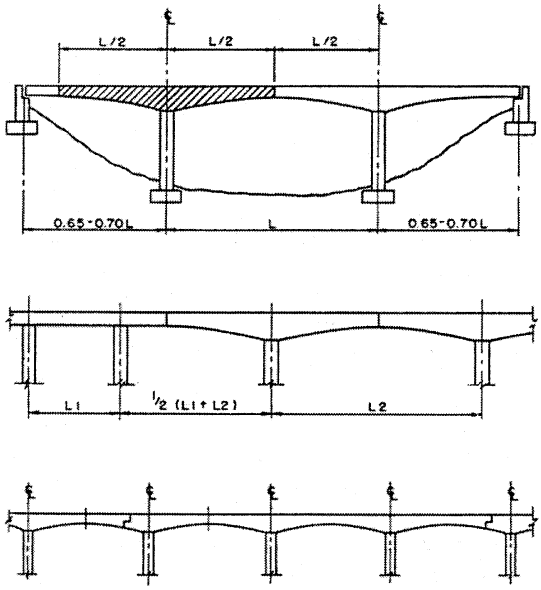


FIGURE 11.34 Balanced cantilever span arrangement.

be necessary to accommodate the change in length of structure due to creep, shrinkage, and thermal changes. The location of the expansion joint within a span will vary, depending on the method of construction.

For balanced cantilever construction, the expansion joints were initially located at the tip of the cantilevers, which is the middle of the span on the completed structure, for ease of construction. Creep effect under dead load plus post-tensioning drives the tip of the cantilever down, resulting in unacceptable angle break at midspan. This disposition is no longer used. An alternative solution is to place the joint at the point of contraflexure of the equivalent continuous span, thus very effectively reducing the angle of break under creep and live load. However, this technique requires expansion segments at midlength of the cantilever, making construction more difficult. The latest technique goes back to the joint at midspan, but with the addition of a stiffening steel beam across the joint, turning the hinged span into a continuous span with expansion capability. Further refinements are introduced such as the capability of controlling the deflection of the span by vertical jacking on the steel beam during the life of the bridge. This technique has been successfully used as it does not interfere with the cantilever erection process (see Figure 11.35).

For spans built with the use of the span-by-span method the expansion joints are typically located at the centerline of a column. The adjacent box-girder spans are both supported by the column with movement allowed between the spans. With this method, the angle break at the expansion joint is minimized, and there is no requirement for temporary moment restraint between the adjacent sections.

11.7.2 Cross-Section Dimensions

11.7.2.1 Overall Box-Girder Dimensions

The overall width of a concrete segmental box-girder bridge is quite adaptable to any requirement. Box-girder spans have been built with widths as low as 3.6 m and as great as 27.50 m, with the configuration of the box girder varying significantly.

The depth of precast segmental box girders is generally somewhat greater than that of similar spans with cast-in-place construction. This increased depth is necessary to offset more stringent requirements for extreme fiber axial stresses and restrictions on the locations of post-tensioning

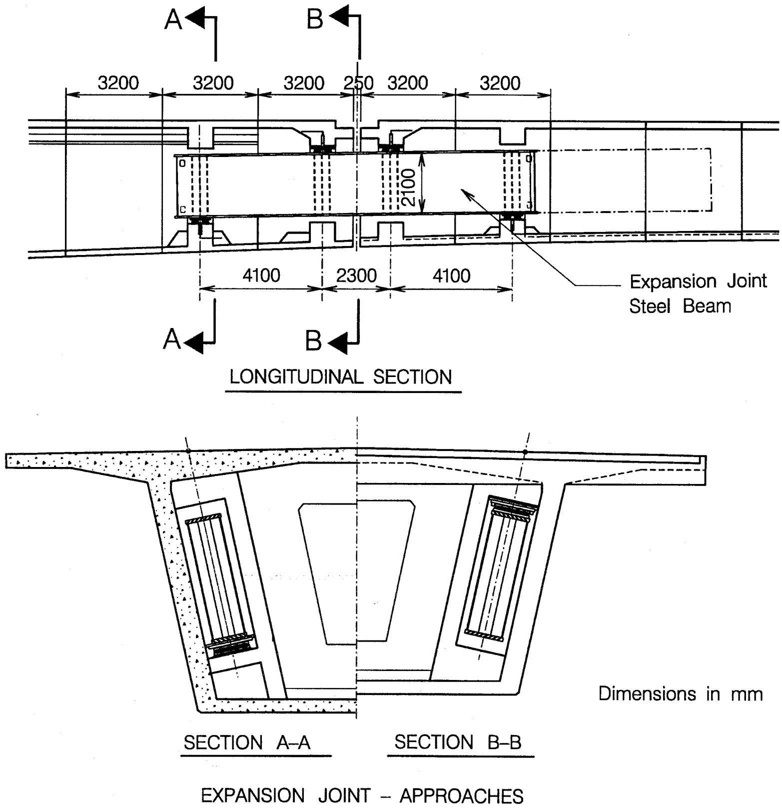


FIGURE 11.35 Expansion joint — approaches.

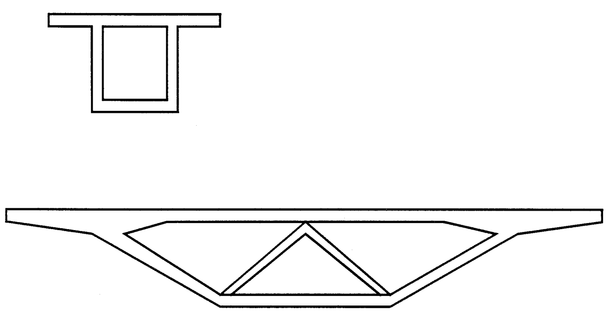


FIGURE 11.36 Various cross sections.

tendons. Multiple-cell, box-girder bridges will also have more webs to place tendons than a comparable width, single-cell segmental box girder. For span-by-span construction, the span-to-depth ratio should not exceed 25 to 1, and is more comfortable at 20 to 1. For balanced cantilever construction, the span-to-depth ratio at the support should not exceed 18 to 1. However, variable-depth box girders built in balanced cantilever fashion are quite common with straight haunched sections and parabolic extrados. The span-to-depth ratio at midspan of a variable-depth balanced cantilever bridge should not exceed 40 to 1 (see Figure 11.36).

11.7.2.2 Web Thickness

The thickness of the web is generally determined such that the required post-tensioning tendons may be placed without interfering with concrete placement or risking cracking during stressing of tendons. Principal stress values at service limit state for no cracking in concrete should be checked in the webs at the neutral axis and at the intersection with top and bottom flanges. This will give a good indication whether the thickness of the web is sufficient. Most design codes also place a limit on the ultimate shear capacity for a box girder to ensure that the web does not fail in diagonal compression prior to the yielding of stirrup reinforcing.

11.7.2.3 Slab Thickness

Slab thicknesses are generally determined to limit deflection under live loading and to provide the necessary flexural capacity. These limits are similar to those of slab thickness for bridge structures built with the use of more traditional construction methods. Span-to-thickness ratios should be in the range of 30 to 1. Since most segmental box girders have transversely post-tensioned top slabs, the minimum thickness of a top slab should be 200 mm, with possibly thicker values at the tendon anchorages. Bottom slab thickness may be less, down to 180 mm, if there is no longitudinal or transverse post-tensioning embedded in the slab.

11.7.3 Temperature Gradients

11.7.3.1 Linear Temperature Gradients

Temperature gradients are caused by the top or bottom surface of the structure being warmer than the other. The shape of the temperature distribution along the depth of the section is beyond the scope of this text. However, this distribution may be assumed to be linear or nonlinear with magnitudes given in relevant texts [3]. Due to its high thermal mass, concrete structures are more adversely affected by the thermal gradient than steel structures.

Effects of a linear temperature gradient can be easily evaluated using hand-calculation methods. Once the magnitude of the temperature gradient has been determined, the unrestrained curvature at any point along the span can be determined by

$$R = \frac{\Delta T \cdot \alpha \cdot E_c}{h} \tag{11.1}$$

where
R = radius of curvature
ΔT = linear temperature differential between top and bottom fibers of cross section
E_c = Modulus of elasticity of concrete
α = thermal expansion coefficient
h = depth of cross section

Once the unrestrained curvature along the structure is known, the final force distribution can be determined by evaluating the redundant support reactions. It is noted that for a statically determinate structure the linear temperature gradient results in zero effect on the structure.

11.7.3.2 Nonlinear Temperature Gradients

Nonlinear temperature gradients are more difficult to evaluate and are best handled by a well-suited computer program. The general theory is presented here; for a more detailed elaboration see Reference [1]. The nonlinear temperature distribution is determined by field measurements and thermodynamic principles. The general shape may be as shown in Figure 11.37. Assuming that the material has linear stress–strain properties, that plane sections will remain plane (Navier–Bernoulli hypothesis), and that temperature varies only with depth (two-dimensional problem), one can make the following theoretical derivation of the problem: the free thermally induced strain is proportional to the temperature distribution; however, this strain distribution violates the second assumptions above, namely, that plane sections remain plane. In order for the section to remain plane under the

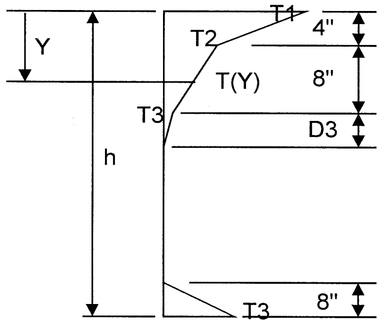


FIGURE 11.37 Nonlinear gradient.

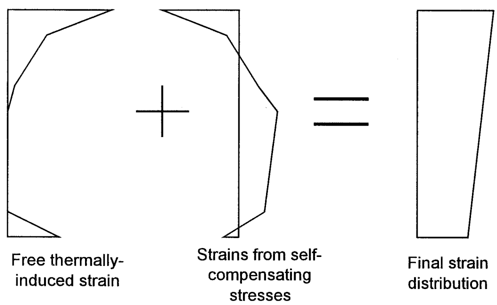


FIGURE 11.38 Self-compensating stresses.

effects of the applied temperature gradient, there must be some induced stress on said section. This is termed self-compensating stress. The final strain distribution on the section is, therefore, linear and is the sum of the free thermally induced strain and the strain induced by the self-compensating stresses (see Figure 11.38).

The self-compensating stresses can be derived as

$$\sigma(Y) = E_c \cdot \sigma \cdot T(Y) - \frac{P}{A} - \frac{M \cdot Y}{I} \qquad (11.2)$$

where

Y = variable along depth of cross section
$T(Y)$ = temperature at abscissa Y
P = $\int_Y E \cdot a \cdot T(Y) \cdot b(Y) dY$
M = $\int_Y E \cdot a \cdot T(Y) \cdot b(Y) \cdot Y \cdot dY$
$b(Y)$ = width of section at abscissa Y

Similar to the linear gradient, there is now a free unrestrained curvature of the structure along its length. If the structure is continuous, this will result in reactions due to the restraint of the system. The unrestrained curvature at any point along the structure is

$$R = \frac{M}{E_c \cdot I} \qquad (11.3)$$

Once the unrestrained curvature along the structure is known, the continuity force distribution can be determined by evaluating the redundant support reactions. The total stress on a section is, therefore, the summation of the self-compensating stresses and the continuity stresses. For a statically determinate structure, the stress on a section is not zero as for a linear temperature gradient; the continuity stresses are zero, but the self-compensating stresses may be significant.

11.7.4 Deflection

11.7.4.1 Dead Load and Creep Deflection

Global vertical deflections of segmental box-girder bridges due to the effects of dead load and post-tensioning as well as the long-term effect of creep are normally predicted during the design process by the use of a computer analysis program. The deflections are dependent, to a large extent, on the method of construction of the structure, the age of the segments when post-tensioned, and the age of the structure when other loads are applied. It can be expected, therefore, that the actual deflections of the structure would be different from that predicted during design due to changed assumptions. The deflections are usually recalculated by the contractor's engineer, based on the actual construction sequence.

11.7.4.2 Camber Requirements

The permanent deflection of the structure after all creep deflections have occurred, normally 10 to 15 years after construction, may be objectionable from the perspective of riding comfort for the users or for the confidence of the general public. Even if there is no structural problem with a span with noticeable sag, it will not inspire public confidence. For these reasons, a camber will normally be cast into the structure so that the permanent deflection of the bridge is nearly zero. It may be preferable to ignore the camber, if it is otherwise necessary to cast a sag in the structure during construction.

11.7.4.3 Global Deflection Due to Live Load

Most design codes have a limit on the allowable global deflection of a bridge span due to the effects of live load. The purpose of this limit is to avoid the noticeable vibration for the user and minimize the effects of moving load impact. When structures are used by pedestrians as well as motorists, the limits are further tightened.

11.7.4.4 Local Deflection Due to Live Load

Similar to the limits of global deflection of bridge spans, there are also limitations on the deflection of the local elements of the box-girder cross section. For example, the AASHTO Specifications limit the deflection of cantilever arms due to service live load plus impact to 1/300 of the cantilever length, except where there is pedestrian use [1].

11.7.5 Post-Tensioning Layout

11.7.5.1 External Post-Tensioning

While most concrete bridges cast on falsework or precast beam bridges have utilized post-tensioning in ducts which are fully encased in the concrete section, other innovations have been made in precast segmental construction. Especially prevalent in structures constructed using the span-by-span method, post-tensioning has been placed inside the hollow cell of the box girder but not encased in concrete along its length. This is know as external post-tensioning. External post-tensioning is easily inspected at any time during the life of the structure, eliminates the problems associated with internal tendons, and eliminates the need for using expensive epoxy adhesive between precast segments. The problems associated with internal tendons are (1) misalignment of the tendons at segment joints, which causes spalling; (2) lack of sheathing at segment joints; and (3) tendon pull-through on spans with tight curvature (see Figure 11.39). External prestressing has been used on many projects in Europe, the United States, and Asia and has performed well.

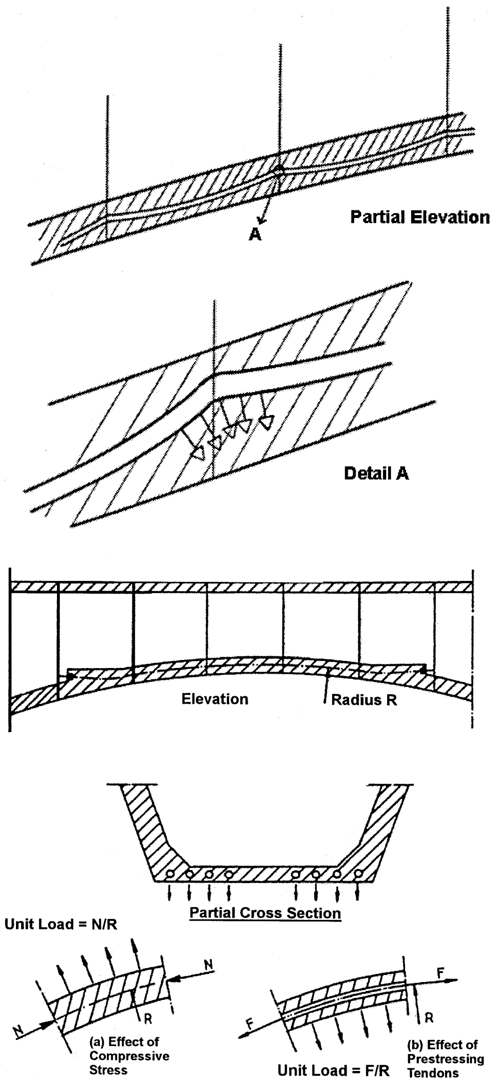


FIGURE 11.39 Problems with internal tendons.

11.7.5.2 Future Post-Tensioning

The provision for the addition of post-tensioning in the future in order to correct unacceptable creep deflections or to strengthen the structure for additional dead load, i.e., future wearing surface, is now required by many codes. Of the positive and negative moment post-tensioning, 10% is reasonable. Provisions should be made for access, anchorage attachment, and deviation of these additional tendons. External, unbonded tendons are used so that ungrouted ducts in the concrete are not left open.

11.8 Seismic Considerations

11.8.1 Design Aspects and Design Codes

Due to typical vibration characteristics of bridges, it is generally accepted that under seismic loads, some portion of the structure will be allowed to yield, to dissipate energy, and to increase the period of vibration of the system. This yielding is usually achieved by either allowing the columns to yield plastically (monolithic deck/superstructure connection), or by providing a yielding or a soft bearing system [6].

The same principles also apply to segmental structures, i.e., the segmental superstructure needs to resist the demands imposed by the substructure. Very few implementations of segmental structures are found in seismically active California, where most of the research on earthquake-resistant bridges is conducted in the United States. The Pine Valley Creek Bridge, Parrots Ferry Bridge, and Norwalk/El Segundo Line Overcrossing, all of them being in California, are examples of segmental structures; however, these bridges are all segmentally cast in place, with mild reinforcement crossing the segment joints.

Some guidance for the seismic design of segmental structures is provided in the latest edition of the AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges [2], which now contains a chapter dedicated to seismic design. The guide allows precast-segmental construction without reinforcement across the joint, but specifies the following additional requirements for these structures:

- For Seismic Zones C and D [1], either cast-in-place or epoxied joints are required.
- At least 50% of the prestress force should be provided by internal tendons.
- The internal tendons alone should be able to carry 130% of the dead load.

For other seismic design and detailing issues, the reader is referred to the design literature provided by the California Department of Transportation, Caltrans, for cast-in-place structures [5-8].

11.8.2 Deck/Superstructure Connection

Regardless of the design approach adopted (ductility through plastic hinging of the column or through bearings), the deck/superstructure connection is a critical element in the seismic resistant system. A brief description of the different possibilities follows.

11.8.2.1 Monolithic Deck/Superstructure Connection

For the longitudinal direction, plastic hinging will form at the top and bottom of the columns. Since most of the testing has been conducted on cast-in-place joints, this continues to be the preferred option for these cases. For short columns and for solid columns, the detailing in this area can be readily adapted from standard Caltrans practice for cast-in-place structures, as shown on Figure 11.40. The joint area is then essentially detailed so it is no different from that of a fully cast-in-place bridge. In particular, a Caltrans requirement for positive moment reinforcement over the pier can be detailed with prestressing strand, as shown below. For large spans and tall columns, hollow column sections would be more appropriate. In these cases, care should be taken to confine the main column bars with closely spaced ties, and joint shear reinforcement should be provided according to Reference [3 or 7].

The use of fully precast pier segments in segmental superstructures would probably require special approval of the regulating government agency, since such a solution has not yet been tested for bridges and is not codified. Nevertheless, based upon first principles, and with the help of strut–tie models, it is possible to design systems that would work in practice [6]. The segmental superstructure should be designed to resist at least 130% of the column nominal moment using the strength reduction factors prescribed in Ref. [2].

Of further interest may be a combination of precast and cast-in-place joint as shown in Figure 11.41, which was adapted from Ref. [8]. Here, the precast segment serves as a form for the cast-in-place portion that fills up the remainder of the solid pier cap. Other ideas can also be derived from the building industry where some model testing has been performed. Of particular interest for bridges could be a system that works by leaving dowels in the columns and supplying the precast segment with matching formed holes, which are grouted after the segment is slipped over the reinforcement [9].

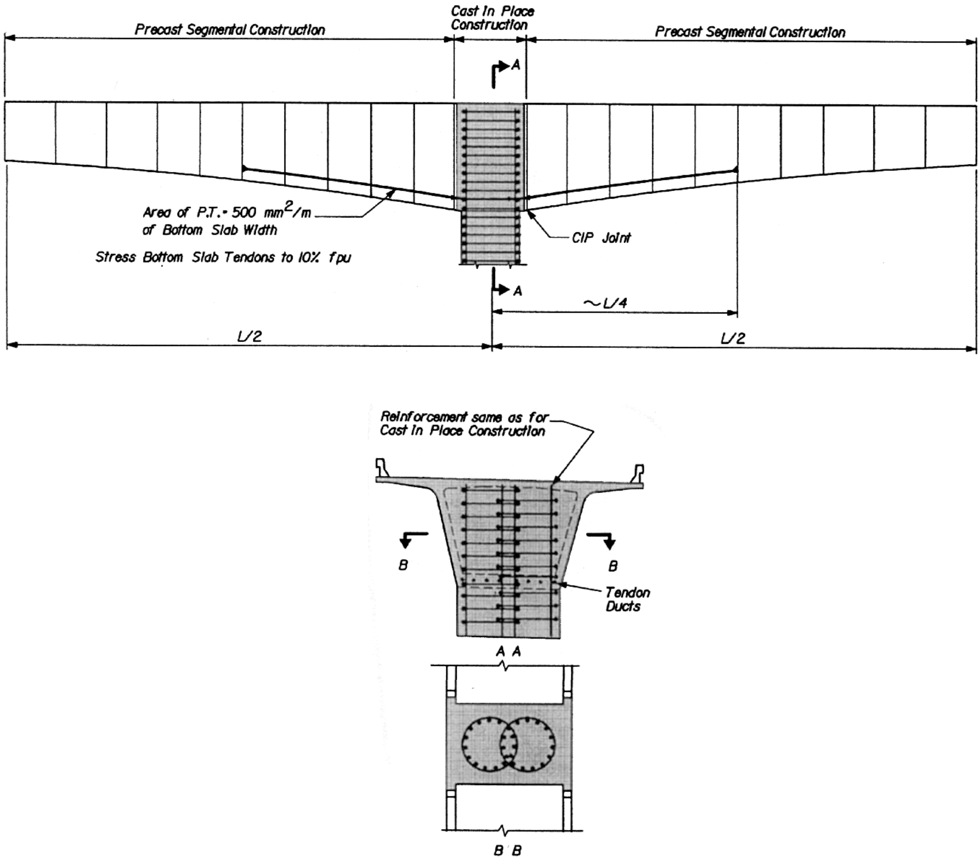


FIGURE 11.40 Deck/pier connection with cast-in-place joint.

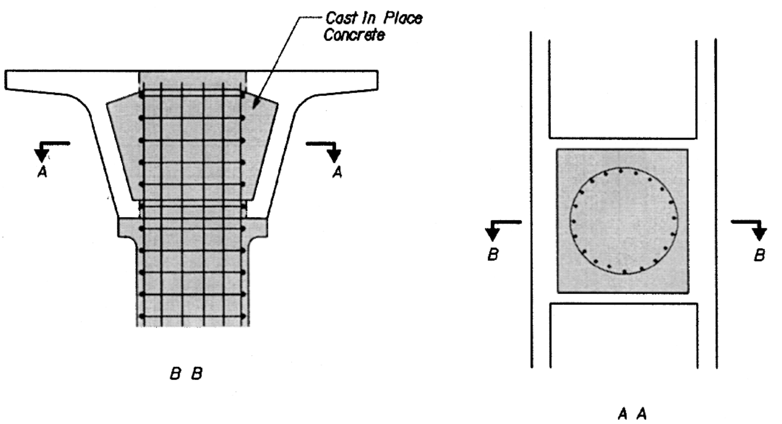


FIGURE 11.41 Combination of precast and cast-in-place joint.

11.8.2.2 Deck/Superstructure Connection via Bearings

Typically, for spans up to 45 m erected with the span-by-span method, the superstructure will be supported on bearings. For action in the longitudinal direction, elastomeric or isolation bearings are preferred to a fixed-end/expansion-end arrangement, since these better distribute the load

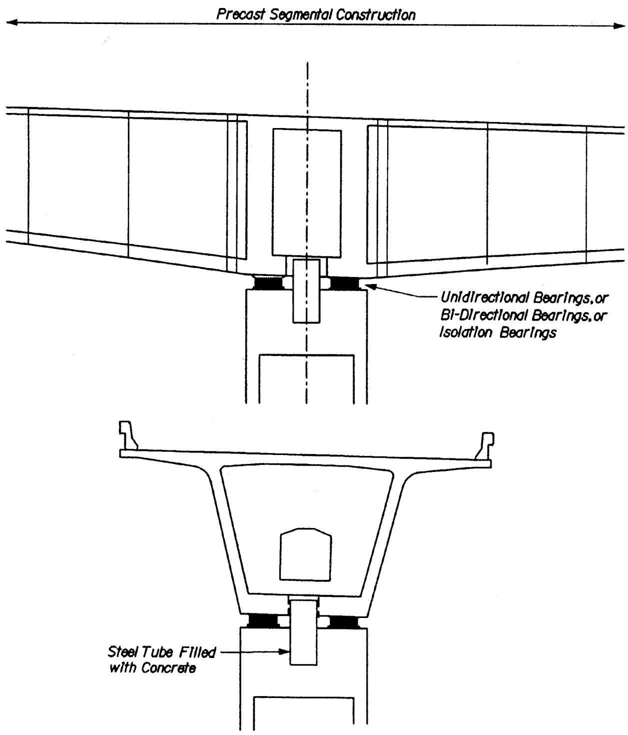


FIGURE 11.42 Deck/pier connection with bearings.

between the bearings. Furthermore, these bearings will increase the period of the structure, which results in an overall lower induced force level (beneficial for higher-frequency structures), and isolation bearings will provide some structural damping as well.

In the transverse direction, the bearings may be able to transfer load between super- and substructure by shear deformation; however, for the cases where this is not possible, shear keys can be provided as is shown in Figure 11.42. It should be noted that in regions of high seismicity, for structures with tall piers or soft substructures, the bearing demands may become excessive and a monolithic deck–superstructure connection may become necessary.

For the structure-on-bearings approach, the force level for the superstructure can be readily determined, since once the bearing demands are obtained from the analysis, they can be applied to the superstructure and substructure. The superstructure should resist the resulting forces at ultimate (using the applicable code force-reduction factors), whereas the substructure can be allowed to yield plastically if necessary.

11.8.2.3 Expansion Hinges

From the seismic point of view, it is desirable to reduce the number of expansion hinges (EH) to a minimum. If EHs are needed, the most beneficial location from the seismic point of view is at midspan. This can be explained by observing Figure 11.43, where the superstructure bending moments, resulting from column plastic hinging (M_p), have been plotted for the case of an EH at midspan and for an EH at quarterspan. For the latter, it can be seen that the moment at the face of the column varies within the range of ±¾ M_p, whereas with the hinge at midspan, the values are only between ±½ M_p.

The location of expansion hinges within a span, and its characteristics, depends also on the stiffness of the substructure and the type of connection of the superstructure to the piers. Table 11.1 presents general guidelines intended to assist in the selection of location of expansion hinges.

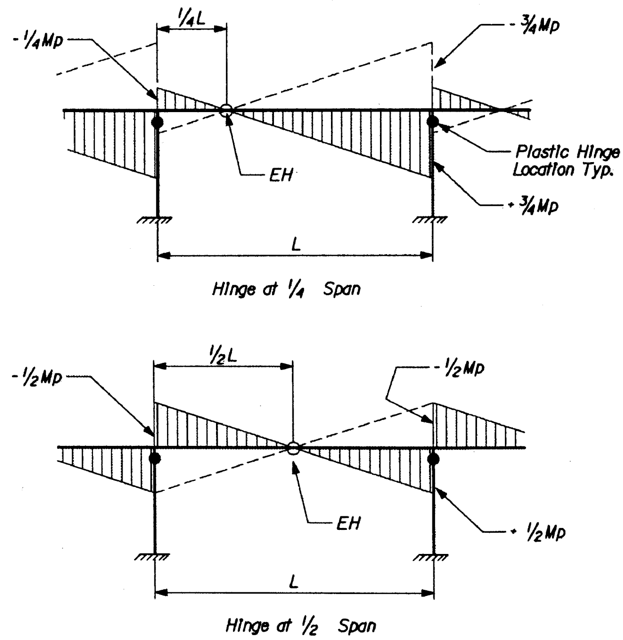


FIGURE 11.43 Longitudinal superstructure seismic moments with hinges at quarterspan and at midspan.

TABLE 11.1 Location of Expansion Hinges in Segmental Bridges

Span Support System	Location of EH		
	Over Pier	Intermediate Point	Midspan
On bearings	• Standard solution for simple spans • For continuous spans generates moderate superstructure moments at adjacent piers	• Complicated erection for cantilever construction • Generates moderate superstructure moments at adjacent piers • Moderate EH openings requiring restrainers and moderate seat widths, or lock-up devices	• Simplest location for cantilever construction • Will require continuity beam inside cross section • Minimizes superstructure seismic moments • Moderate EH openings requiring adequate gap between the end segments
Monolithic with pier	Not applicable	• Complicated erection for cantilever construction • Generates very large superstructure moments at adjacent piers • If substructure is stiff, expect relatively small EH movements; otherwise expect very large movements, requiring restrainers and large seat widths, or lock-up devices	• Simplest location for cantilever construction • Will require continuity beam inside cross section • Minimizes superstructure seismic moments • If substructure is stiff, expect relatively small EH movements; otherwise expect very large movements, requiring lock-up devices

11.8.2.4 Precast Segmental Piers

Precast segmental piers are usually hollow cross section to save weight. From research in other areas it can be extrapolated that the precast segments of the pier would be joined by means of unbonded prestressing tendons anchored in the footing. The advantage of unbonded over bonded tendons is that for the former, the prestress force would not increase significantly under high column displacement demands, and would therefore not cause inelastic yielding of the strand, which would otherwise lead to a loss of prestress.

The detail of the connection to the superstructure and foundation would require some insight into the dynamic characteristics of such a connection, which entails joint opening and closing — providing that dry joints are used between segments. This effect is similar to footing rocking, which is well known to be beneficial to the response of a structure in an earthquake. This is due to the period shift and the damping of the soil. The latter effect is clearly not available to the precast columns, but the period shift is. Details need to be developed for the bearing areas at the end of the columns, as well as the provision for clearance of the tendons to move relative to the pier during the event.

If the upper column segment is designed to be connected monolithically to the superstructure, yielding of the reinforcement should be expected. In this case, the expected plastic hinge length should be detailed ductile, using closely spaced ties [3,5].

11.9 Casting and Erection

11.9.1 Casting

There are obvious major differences in casting and erection when working with cast-in-place cantilever in travelers or in handling precast segments. There are also common features, which must be kept in mind in the design stages to keep the projects simple and thereby economic and efficient, such as

- Keeping the length of segments equal and segments straight, even in curved bridges;
- Maintaining constant cross section dimensions as much as possible;
- Minimizing the number of diaphragms and stiffeners, and avoiding dowels through formwork.

11.9.1.1 Cast-in-Place Cantilevers

Conventional Travelers

The conventional form traveler supports the weight of the fresh concrete of the new segment by means of longitudinal beams or frames extending out in cantilever from the last segment. These beams are tied down to the previous segment. A counterweight is used when launching the traveler forward. The main beams are subjected to some deflections, which may produce cracks in the joint between the old and new segments. Jacking of the form during casting is sometimes needed to avoid these cracks. The weight of a traveler is about 60% of the weight of the segment. The rate of construction is typically one segment per traveler per week. Precast concrete anchor blocks are used to speed up post-tensioning operations. In cold climates, curing can be accelerated by various heating processes.

Construction Camber Control

The most critical practical problem of cast-in-place construction is deflection control. There are five categories of deflections during and after construction:

- Deflection of traveler frame under the weight of the concrete segment;
- Deflection of the concrete cantilever arm during construction under the weight of segment plus post-tensioning;
- Deflection of cantilever arms after construction and before continuity;
- Short- and long-term deflections of the continuous structure;
- Short- and long-term pier shortenings and foundation settlements.

The sum of the various deflection values for the successive sections of the deck allows the construction of a camber diagram to be added to the theoretical profile of the bridge. A construction camber for setting the elevation of the traveler at each joint must also be developed.

11.9.1.2 Precast Segments

Opposite to the precast girder concept where the bridge is cut longitudinally in the precast segmental methods, the bridge is cut transversally, each slice being a segment. Segments are cast in a casting yard one at a time. Furthermore, the new segment is cast against the previously cast segment so that the faces in contact match perfectly. This is the match-cast principle. When the segments are reassembled at the bridge site, they will take the same relative position with regard to the adjacent segments that they had when they were cast. Accuracy of segment geometry is an absolute priority, and adequate surveying methods must be used to ensure follow-up of the geometry.

Match casting of the segments is a prerequisite for the application of glued joints, achieved by covering the end face of one or both of the meeting segments with epoxy at the erection. The epoxy serves as a lubricant during the assembly of the segments, and it ensures a watertight joint in the finished structure. Full watertightness is needed for corrosion protection of internal tendons (tendons inside the concrete). The tensile strength of the epoxy material is higher than that of the concrete, but, even so, the strength of the epoxy is not considered in the structural behavior of the joint. The required shear capacity is generally provided by shear keys, single or multiple, in combination with longitudinal post-tensioning.

With the introduction of external post-tensioning, where the tendons are installed in PE ducts, outside the concrete but inside the box girder, the joints are relieved of the traditional requirement of watertightness and are left dry. The introduction of external tendons in connection with dry joints greatly enhanced the efficiency of precasting.

11.9.1.3 Casting Methods

There are two methods for casting segments. The first one is the long-line method, where all the segments are cast in their correct position on a casting bed that reproduces the span. The second method, used most of the time, is the short-line method, where all segments are cast in the same place in a stationary form, and against the previously cast segment. After casting and initial curing, the previously cast segment is removed for storage, and the freshly cast segment is moved into place (see Figure 11.44).

11.9.1.4 Geometry Control

A pure translation of each segment between cast and match-cast position results in a straight bridge (Figure 11.45). To obtain a bridge with a vertical curve, the match-cast segment must first be translated and given a rotation α in the vertical plane (Figure 11.46). Practically, the bulkhead is left fixed and the mold bottom under the conjugate unit adjusted. To obtain a horizontal curvature, the conjugate unit is given a rotation β in the horizontal plane (see Figure 11.47). To obtain a variable superelevation, the conjugate unit is rotated around a horizontal axis located in the middle of the top slab (Figure 11.48).

All these adjustments of the conjugate unit can be combined to obtain the desired geometry of the bridge.

11.9.2 Erection

The type of erection equipment depends upon the erection scheme contemplated during the design process; the local conditions, either over water or land; the speed of erection and overall construction schedule. It falls into three categories, independent lifting equipment such as cranes, deck-mounted lifting equipment such as beam and winch or swivel crane, and launching girder equipment.

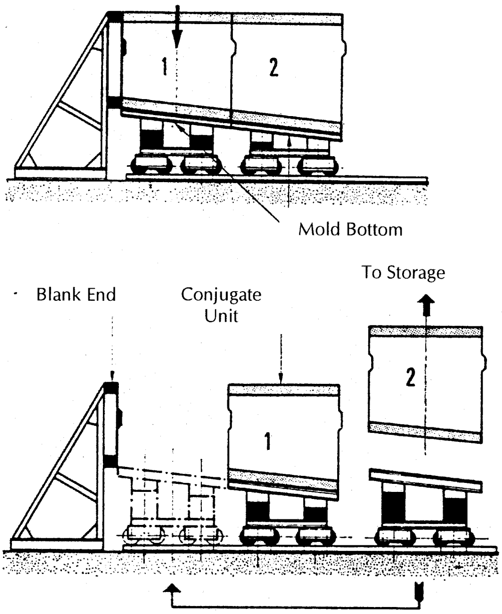


FIGURE 11.44 Typical short-line precasting operation.

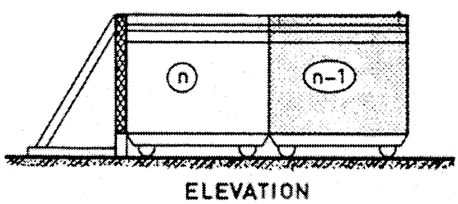


FIGURE 11.45. Straight bridge.

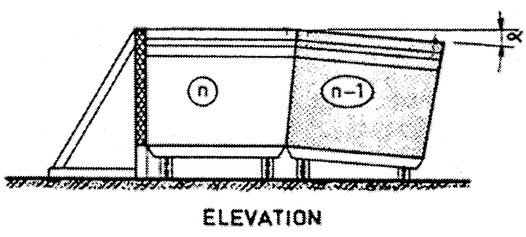


FIGURE 11.46 Bridge with vertical curve.

11.9.2.1 Balanced Cantilever Method

The principle of the method is to erect or cast the pier segment first, then to place typical segments one by one from each side of the pier, or in pairs simultaneously from both sides. Each newly placed precast segment is fixed to the previous one with temporary PT bars, until the cantilever tendons are installed and stressed. The closure joint between cantilever tips is poured in place and continuity tendons installed and stressed.

In order to carry out this erection scheme, segments must be lifted and installed at the proper location. The simplest way is to use a crane, either on land or barge mounted. Many bridges have

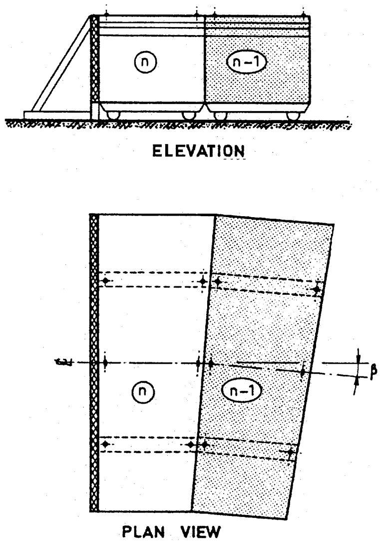


FIGURE 11.47 Bridge with horizontal curve.

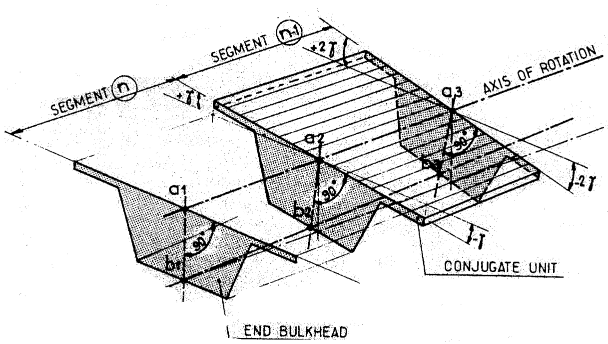


FIGURE 11.48 Bridge with superelevation.

been erected with cranes as they do not require an investment in special lifting equipment. This method is slow. Typically, two to four segments per day are placed. It is used on relatively short bridges. An alternative is to have a winch on the last segment erected. The winch is mounted on a beam fixed to the segment. It picks up segments from below, directly from truck or barge. After placing the segment, the beam and winch system is moved forward to pick up the next segment and so on. Usually, a beam-and-winch system is placed on each cantilever tip. This method is also slow; however, it does not require a heavy crane on the site, which is always very expensive, especially if the segments are heavy.

When bridges are long and the erection schedule short, the best method is the use of launching girders, which then take full advantage of the precast segmental concept for speed of erection.

There are two essential types of self-launching gantries developed for this erection method. The first type is a gantry with a length slightly longer than the typical span (see Figure 11.49). During erection of the cantilever, the center leg rests on the pier while the rear leg rests on the cantilever tip of the previously erected span, which must resist the corresponding reaction. Prior to launching, the back spans must be made continuous. Then, the center leg is moved to the forward cantilever

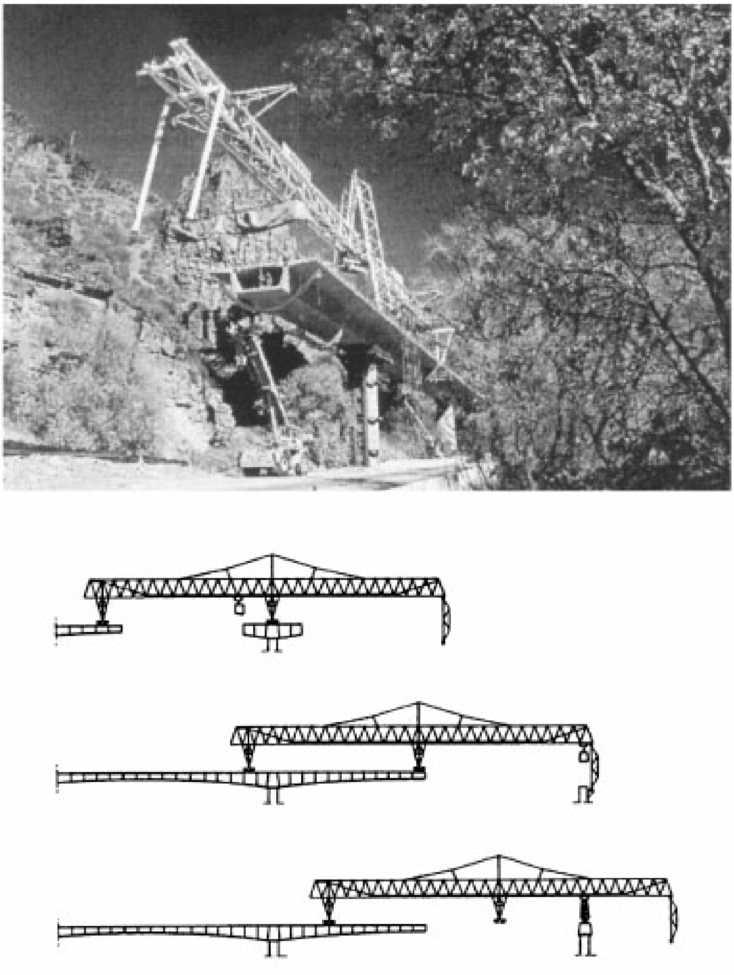


FIGURE 11.49 French Creek Viaduct, U.S.— single erection truss with portal legs.

tip, which must resist the weight of the gantry plus the weight of the pier segment. This stage controls the design of the gantry, which must be made as light as possible, and of the cantilever.

The second type of gantry has a length that is twice that of the typical span (see Figure 11.50). The reaction from the legs during the erection and launching of the next span is always applied on the piers, so there is no concentrated erection load on the cantilever tip. Each erection cycle consists of the erection of all typical segments of the cantilever and then the placement of the pier segment for the next cantilever, without changing the position of the truss.

The gantries can be categorized by their cross section: single truss, with portal-type legs, and two launching trusses with a gantry across. The twin box girders of the bridge in Hawaii were built with two parallel, but independent trusses (see Figure 11.51), with a typical span of 100.0 m, segment weights of 70 tons; the two bridge structures are 27.5 m apart with different elevations and longitudinal slopes. This system is a refinement of the first type of gantry applied to twin decks with variable geometry.

Normally, the balanced cantilever method is used for spans from 60 to 110 m, with a launching girder. One full, typical cycle of erection is placing segments, installing and stressing post-tensioning tendons, and launching the truss to its next position. It takes about 7 to 10 days, but may vary greatly according to the specifics of a project and the sophistication of the launching girder. With proper equipment and planning, erection of 16 segments per day has been achieved.

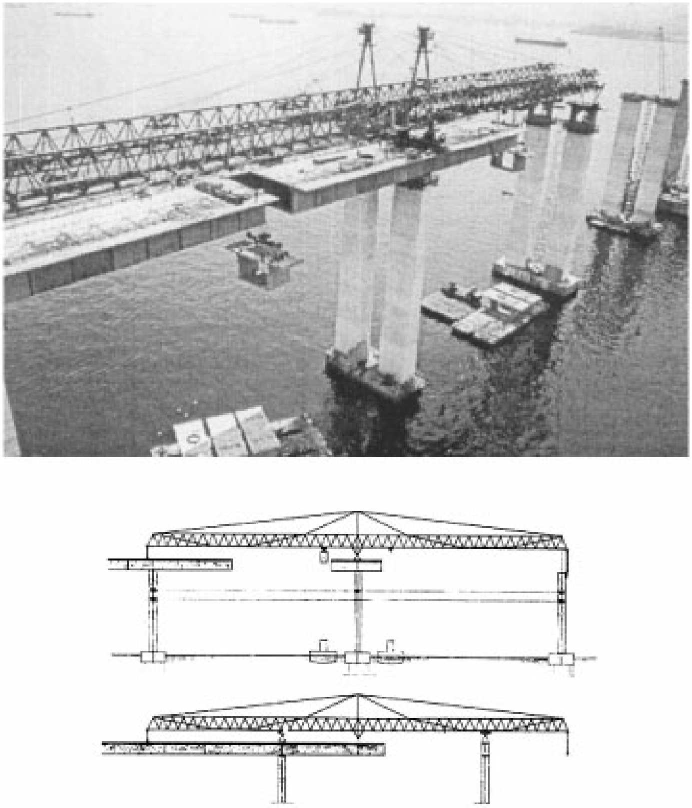


FIGURE 11.50 Río Niteroi, Brazil — two segments being erected simultaneously from one erection truss.

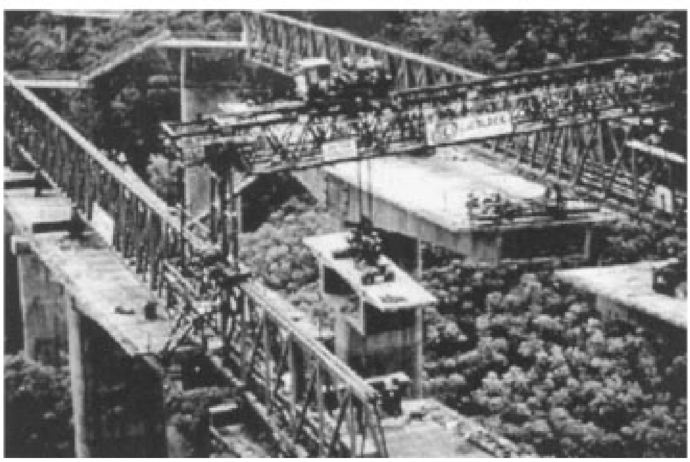


FIGURE 11.51 H-3 Windward Viaduct, Hawaii.

A modification of this method was used to build the 13-km-long Confederation Bridge (typical span 250 m long), linking Prince Edward Island with New Brunswick in Atlantic Canada. The main girder was constructed in a precasting yard as a precast, balanced cantilever. Then the 197-m-long main girder, with a self-weight of 7500 tons, was placed on the pier with a floating crane (see Figure 11.52).

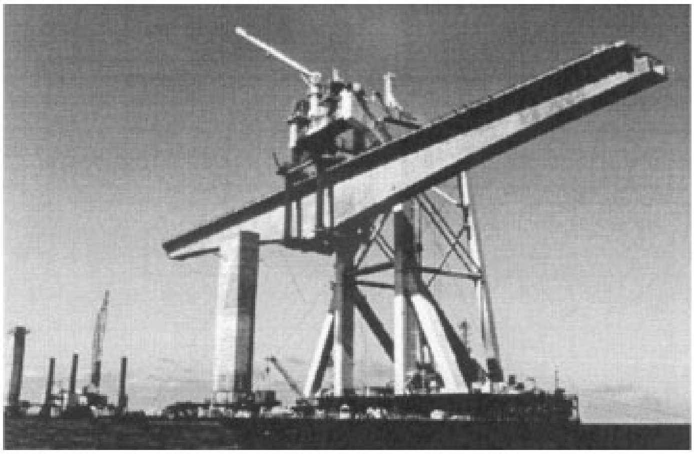


FIGURE 11.52 Confederation Bridge, Canada — floating crane.

FIGURE 11.53 Bang Na–Bang Pli–Bang Pakong Expressway, Thailand — D6 segment erection.

11.9.2.2 Span-by-Span Construction

In the first stage, all precast segments for each span are assembled on an erection girder. The second stage is installing and stressing the tendons, and as a result the span becomes self-supported. In comparison with the balanced cantilever method, those girders have to be designed to carry the load of the entire span. Normally, the duration of one erection cycle is 2 to 3 days per span.

The 56-km-long Bang Na–Bang Pli–Bang Pakong Expressway in Bangkok, carrying six lanes of traffic, totaling 27 m in width, is assembled on erection girders. The girder is placed in the middle of a Y-shaped column. The segments, with self-weight up to 100 tons, are placed on the chassis with a swivel crane and then transported to their final position. Two schemes of erection were developed: a swivel crane mounted to the front of the girder picking up segments from trucks on the highway below and a swivel crane placed on the previously erected span with segments delivered over the deck already built (see Figure 11.53).

Another principle used in erection equipment for the span-by-span method, is the so-called overhang girder. In this case, the girder is above the superstructure, and the precast segments are hung from it.

11.9.2.3 Safety

Due to the inherent character of temporary structures, erection equipment is usually designed to take full advantage of the materials and care must be taken to analyze in-depth all construction stages, anticipating mistakes or shortcuts made on site that always occur, and stay within reasonable safety limits. Overall stability when resting on temporary supports or during launching, reversal of forces, bucking, etc., are the most common problems encountered in these structures. Lifting bars or tie-downs must always be designed with failure or mishandling of one of those elements in mind, and appropriate ultimate resisting paths incorporated in the concept.

11.10 Future of Segmental Bridges

Since their appearance in 1962, precast segmental technologies have been used worldwide in the design and construction of practically all types of bridges. Nevertheless, in the last 5 years, an important further development of these technologies has taken place in Southeast Asia which will have a decisive impact on the way bridges will be built in the next century.

11.10.1 The Challenge

The explosive development of the Southeast Asian economies has been forcing local governments to find new solutions for building infrastructures (build, operate, transfer), which can, when properly managed, become success stories for both the government agencies who organize the projects and for the private groups who develop them.

Privatization of such projects is now accepted by numerous countries as a viable solution for the challenges they face. Large infrastructure projects, worth billions of dollars, are at present being designed, built, financed, and operated by private companies in the region. This trend is expected to extend progressively to the global construction markets. The two key factors of such projects are the amount of toll to be paid by the users and the duration of the concession until the project is transferred to the government agency. For the roadway projects, the tolls vary anywhere from between $1 to $10. The duration of the concessions varies in general from 25 to 35 years.

While basically simple, the scheme presents very complex problems for its implementation. Multidisciplinary skills and a new vision for the design, construction, and operation of the roads and bridges are necessary in order to avoid technical or financial failures. The challenges that private organizations must be able to face can be summarized in just a few words: they need to design and build very competitive projects in the shortest possible time, ensuring the longest possible service life. In light of the experiences gained in the new markets, it is becoming more evident each day that precast segmental technologies often bring the right solutions to these challenges.

11.10.2 Concepts

When Jean Muller invented the precast concrete segmental technology in 1962, his vision was to create an industrialized construction system to build any type of bridge with standard modules, assembled with post-tensioning, without any cast-in-place concrete.

To achieve this objective, he developed the concept of match-cast joints, which allows the transverse slicing of concrete box girders and the assembly of such slices — the segments — in the same order as they were produced, without any need for additional *in situ* concrete to complete the bridge deck [10, 11].

In addition to epoxy-glued joints, the use of dry joints became widespread. In 1978, through the design of the Long Key Bridge in Florida, internal post-tensioning was replaced by external post-tensioning [12]. A number of other concepts invented by Jean Muller allowed further development of the modular construction concept: span-by-span assembly method (Long Key Bridge), progressive

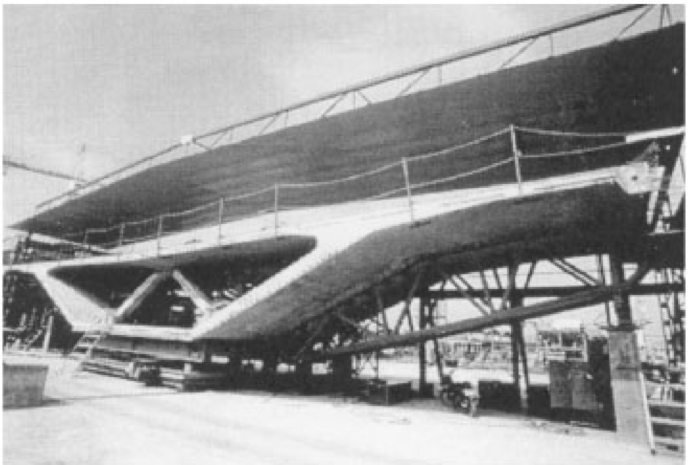


FIGURE 11.54 Bang Na Expressway, Thailand — D6, six-traffic-lane segment at precast yard.

TABLE 11.2 Standard Segments

Segment	Lanes	Widths (m) Min-Max
D2	2	08–12
D3	3	10–15
D4	4	14–20
D6	6	18–30

placing (Linn Cove Bridge), precast segmental construction of the piers, D6 cable-stayed segments (Sunshine Skyway Bridge), delta frames (James River Bridge, C&D Canal Bridge), etc.

These concepts and others developed more recently for the large projects, which are being built in Canada and Thailand, allow the prefabrication of bridge structures with precast modules ranging from 20 tons, in channel-shaped overpasses, to 7500 tons for the main girders of the Confederation Bridge in Canada. In this project, one of the largest bridges ever built, no cast-in-place structural concrete was used in the construction of the main spans [13]. The construction modules are all manufactured in sophisticated and industrialized precasting plants that ensure an unequaled construction rate and quality (see Figure 11.54).

By further standardizing the segments with the number of traffic lanes that they carry, we have developed the modules in Table 11.2. These modules allow the construction of viaducts of any width, ranging from 7 to 8 m up to 30 m. Concurrently, with the effort to standardize the cross sections for precast segmental bridges, there has been significant development of design, shop drawing software, and geometry control systems. This gives us the capability to produce drawings by the thousands for viaducts, interchanges, and merging sections that give, for each segment, the detailed geometry and dimensions of concrete and rebar and the layout of the post-tensioning. Such shop drawings are an essential part of the system and must be integrated into the structural design; no standardization and, hence, no industrialization is possible without them (Figure 11.55).

11.10.3 New Developments

The dynamic business environment of Southeast Asian markets is quite favorable for the introduction of innovative concepts. In recent years, technologies that took over 20 years to develop in Europe and some 10 years to spread throughout the United States were absorbed by countries such

FIGURE 11.55 The Confederation Bridge, Canada — Prince Edward Island precasting yard.

as Thailand, which had limited prior experience in the field of bridge engineering, and already concepts never used before are being developed for new projects, thus giving these countries a leading position in construction innovation. By using the most innovative technologies, the developers involved in the private roadway projects are dramatically changing the very nature of the construction business that, until very recently, was considered one of the most conservative sectors of the industry. The innovative concepts that the industrialization of bridges is introducing cover different areas:

- Reduction of construction time and construction cost;
- Durability of the structures (25 to 50 to 100 years);
- Replaceability of components such as bearings, post-tensioning, stays;
- Earthquake resistance of the structures;
- Staged construction;
- Integrated inspection and surveillance systems;
- Users' comfort and safety.

It is evident that the multiplication of such private projects, where cost, time, and durability are the decisive factors, will open the way to innovation in the bridge business as never before.

11.10.4 Environmental Impact

To prevent private projects from turning into environmental nightmares, private developers need to comply with strict obligations with respect to aesthetics, rights of way, and maintenance of the structures during the duration of the concessions. Government agencies have been developing design, construction, and operation criteria that will progressively become the rules of the BOT projects. As an example, such rules may force structural engineers to conceive structures that can be built in or over crowded areas of cities, with a minimal impact on existing conditions. Or they may impose specific constraints on aesthetics, shapes, or dimensions of structural elements. Further, they may require maintenance costs to be budgeted.

The involvement of the communities in such projects, even if sometimes it may be difficult to manage and may require a profound knowledge of the interests and aspirations of those concerned by the project, is essential for the smooth development of the work. In general, projects that are not well integrated into the context of the local environment or not consistent with the users' expectations run the risk of finishing in disarray or remaining incomplete.

11.10.5 Industrial Production of Structures

The experience acquired in large- or medium-size projects demonstrates that the industrialization of the production of structural elements always brings clear advantages in terms of quality and construction time. What frequently has been less noticed is the advantage that such industrialization can offer as far as the cost of a specific project.

The major change in the contractual conditions of the BOT projects is that the cost of "design + construction time" can now be estimated very precisely. If the completion of a project is delayed by 1 month, for instance, in a project worth U.S.$800,000,000, the cost to the developer is approximately equal to the interest that must be paid on that amount. If the interest is 5%, this represents U.S.$40,000,000/year; thus, every month gained in the duration of the design + construction period represents U.S.$3,300,000, or roughly, U.S.$100,000/day.

In these large projects the industrialization of production and the use of sophisticated systems to transport, erect, and assemble the prefabricated modules is reducing the duration of cycles, which usually may take 6 years when managed by the government agencies, to some 3 years, when managed by private organizations in a fully integrated way.

The introduction of the "assembly-line" approach to bridge building was taken to its limits during the construction of the Northumberland Strait Crossing (Confederation Bridge), Canada, a major bridge project which extends over 13 km of icy strait, with extreme weather conditions. The actual assembly of the components that constitute the 43 spans, 250-m each, took place in only 12 months, whereas to build just a single cast-in-place span of 250 m by traditional means is a difficult venture that takes at least 2 years (see Figure 11.55).

11.10.6 The Assembly of Structures

The production of the structural modules for precast segmental projects represents half of the process. The other half relates to the transport of these modules to the site, to their erection, and to the assembly methods to constitute the structural integrity of the bridge.

Generally, for the transport of current segments weighing from 30 to 100 tons, equipment already available in the market has been used. The transport is commonly by road, using convoys of "low boys," or by water, using barges. In some projects currently being built, 30 to 40 segments weighing between 50 and 60 tons are transported every night from the casting yards situated some 100 km from the large metropolis, to the site in the center of the city. The segments are picked up directly from the trucks by the assembly gantries, between midnight and five o'clock in the morning, to avoid interfering with heavy city traffic during the day.

The erection and assembly of such segments are also performed in a highly industrialized environment. With the span-by-span construction method, spans of 40 m can be assembled in 2 days, with crews working after hours. The cycle is almost independent of the type of segment, from D2 to D6, and therefore, the method is ideal for spans from 30 to 45 m. For cantilever construction, special gantries have been developed to assemble two parallel viaducts mimicking the procedure used in Hawaii, achieving speeds of construction of 3 weeks to complete two double cantilevers of 100 m [14]. This method very competitively covers spans of 80 to 120 m. Progressive placing of segments, using a swivel crane, has also been improved for this type of construction, which allows construction of spans from 45 to 65 m. Finally, for large cable-stayed spans, the use of precast segmental technologies successfully tested in milestone projects, like the Sunshine Skyway and the James River Bridges, is now being developed to cover different cross sections for the segments and to combine space trusses and composite sections [15].

The use of gigantic floating cranes, such as the *Svanen*, to place units as large as 190 m and with weights of 7500 tons, opens new prospects for the construction of bridges over rivers and straits (see Figure 11.52). Bridges that previously were almost impossible to build competitively and within the common constraints of construction schedules can now be conceived, designed, and built in short periods of time, by intensive use of precast technologies.

FIGURE 11.56 View of the completed Second Expressway System Project, Thailand.

Clearly, this evolution is going to accelerate and will become global. Equipment designed to be used anywhere in the world will allow for the reduction of costs charged on a specific project. Furthermore, we can expect improvements in the performance and reliability of equipment specifically conceived to perform heavy lifting and assembly of bridge modules. Bridges will be designed which take into consideration the availability and the characteristics of these machines. Construction methods will then become, more than ever, a decisive factor in the design of structures.

11.10.7 Prospective

Design-and-build projects that were common in the 1960s provided some of the most innovative contributions to bridge engineering. Engineers and contractors working together produced competitive structures that paved the way for the development that has taken place all over the world during the last quarter century (Figure 11.56).

A new wave of innovative bridge concepts is already being generated by the privatization of roadway and bridge projects. This wave, which began in the vibrant business environment of Southeast Asia, will eventually reach the United States and the European markets. This time, engineers and contractors will be seconded by developers, finance specialists, and industrialists to shape the structures that will be built during the next century. The construction industry will also join other key industries in adopting high-technology and innovation as essential ingredients of its renewal [16].

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