# Recommended Practice for Precast Post-Tensioned Segmental Construction


Reported by

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## Synopsis

This Joint PCI-PTI committee report presents basic recommendations for the design and construction of precast post-tensioned concrete structures which are composed of individual segments that are connected by the use of post-tensioning. The recommendations cover the fabrication, transportation, and erection of the precast component segments, details of the construction of joints, tendons and anchors, and design considerations applicable to segmental construction.

It should be emphasized that this report is published as a guide on segmental construction and should not be considered a specification. The report contained herein is an update of the report entitled "Recommended Practice for Segmental Construction in Prestressed Concrete," published in the March-April 1975 PCI JOURNAL.

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# CHAPTER 1 — INTRODUCTION

## 1.1 General

Precast post-tensioned segmental construction is defined as a method of construction for bridges, buildings, tanks, silos, cooling towers, tunnels and other structures in which primary load carrying members are composed of precast concrete segments post-tensioned together.

## 1.2 Historical Background

The concept of dividing precast concrete structural members into smaller segments and connecting them with stressed rods or bars dates back as far as 1888 to 1907 and early ideas about prestressed concrete by Dohring, Mandl, Lund and others.[1]

Five bridges designed in Europe of a segmental precast girder type were constructed between 1946 and 1950. These girders were prestressed longitudinally, vertically and transversely. The 240-ft (73 m) spans were flat two-hinged portal frame structures with adjustable hinged bearings to compensate for shortening due to shrinkage and creep of concrete.[1,2] In 1951, a small number of bridge girders in the United States were built of precast concrete blocks. These blocks were strung onto prestressing cables, joints of the net section were mortared, harped tendons were post-tensioned, and tendons were grouted.[3] In the United States in 1952 a single span girder bridge was constructed by dividing each girder into three precast segments. After curing, the segments were reassembled at the site with cold joints and post-tensioned.[4]

Thirty-two major bridges outside of North America were reviewed (in 1971) in a state-of-the-art paper about segmental bridges.[5] These bridges were built between the years 1960 and 1969. Many design parameters resulted in variations in span, number of segment cells, span-depth ratios, segment lengths, joints, types, and erection methods. Six more bridges in various countries were reviewed in another state-of-the-art paper published in 1975.[4]

In North America well over 50 box girder bridges of segmental construction have either been built or are under construction. In 1977 a cable-stayed precast post-tensioned segmental bridge was completed.[6-8]

The Portland Cement Association library has two annotated bibliographies,[9,10] available for review, which list 195 articles on segmental and cantilever cast-in-place bridge construction between 1962 and 1975.

Although the term segmental has most often been applied to bridge construction the concept of using segmental construction has been successfully utilized in many projects other than bridges. For example: the Mutual Life Building in North Carolina; Gulf Life Tower in Jacksonville, Florida; the Five Points Station — MARTA Rapid Transit, Atlanta, Georgia; the Velodrome and Olympic Stadium structures in Montreal, Canada; and numerous buildings using shells and roof girders.[11,12]

Chapter 2 presents examples of the varied use of precast post-tensioned segmental construction.

## 1.3 Limitations of this Recommended Practice

This report is a recommended practice on segmental construction, not a building code or specification. It is presented as a guide to the design and construction of structures using segmental elements to insure safety and serviceability. Engineering judgment must be applied to these recommendations.

## 1.4 Scope

These recommendations are intended to cover those conditions which affect segmentally constructed members differently from non-segmentally constructed members. They consider the fabrication, transportation, and erection of the precast component segments, the construction of joints, tendons and anchors, and design considerations applicable to segmental construction.

## 1.5 Definitions

*Adhesive* — A bonding material used in joints.

*Anchorage* — The means by which the prestressing force is transmitted from the prestressing steel to the concrete.

*Coating* — Material used to protect the prestressing steel against corrosion and/or provide lubrication.

*Couplers* — Hardware to transmit the prestressing force from one partial length prestressing tendon to another.

*Grout* — A mixture of cement, and water with or without admixtures. (In some instances sand is used as a filler).

*Joint* — The region of the structure between interfaces of precast segments with or without adhesive or grouting materials and with or without reinforcement.

*Prestressing Steel* —The part of a post-tensioning tendon which is elongated and anchored to provide the necessary permanent prestressing force.

*Segment* — A precast element made out of concrete, with or without reinforcement, prestressed or non-prestressed.

*Sheathing or Duct* — Enclosure around the prestressing steel.

*Tendon* — The complete assembly for post-tensioning, consisting of anchorages, couplers, and prestressing steel with sheathing.

# CHAPTER 2 — EXAMPLES OF SEGMENTAL CONSTRUCTION

## 2.1 General

Many structures have been designed and built using some variation of the concept of segmental construction. The following examples are only a small portion of those built. These were recently reported in technical journals.

## 2.2 Airport Control Tower

In 1973 a 180-ft (54.86 m) high airport control tower[13] consisting of four service cores was segmentally constructed using box segments approximately 10 ft (3 m) square in plan and 7½ ft (2.3 m) high. The four cores are topped by a 16-ft

Fig. 1(a). Airport control tower. Structural steel platforms were set at 15-ft (4.57 m) intervals to connect the four service cores.

Fig. 1(b). Erection of precast box segments. Note that the modules were stacked to 180-ft (54.9 m) height.

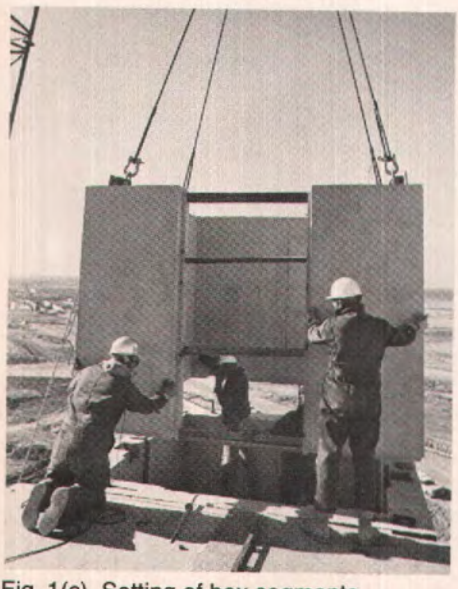


Fig. 1(c). Setting of box segments.

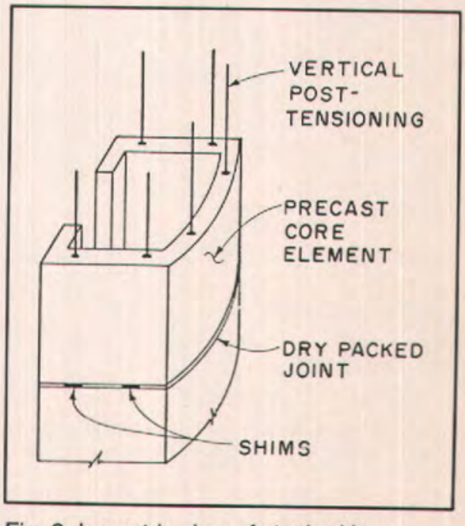



Fig. 2. Isometric view of stacked box segments. Sketch shows the major components of segment.

Fig. 3. Erection of precast panel for liquid natural gas storage tank.

(4.88 m) high steel cab housing the control tower observation deck and facilities. Vertical post-tensioning was used to stabilize the core segments during construction and provide moment and shear capacity to resist service loads. Fig. 1 shows the tower during construction. Fig. 2 is an isometric view of the stacked and post-tensioned box segments.

## 2.3 Liquid Natural Gas Storage Tanks

In 1974 in the United States, two 900,000 bbl liquid natural gas storage tanks[14] were completed using 8 x 60 ft (2.44 x 18.3 m) high precast wall panels. After completion of erection of the panels the structure was post-tensioned circumferentially. The tanks are composed of two concentric walls, the outer wall

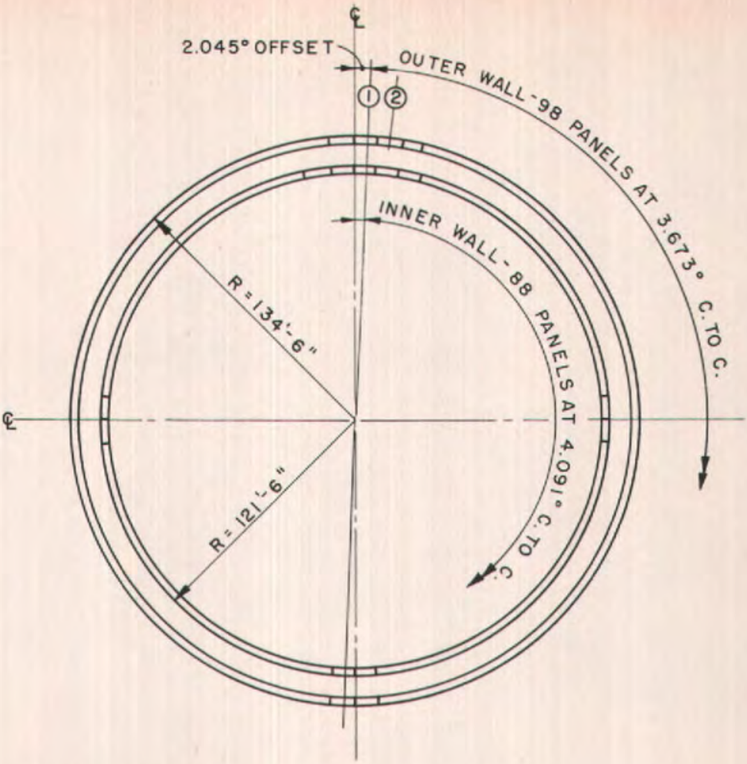



Fig. 4a. Plan of liquid natural gas tank.

having a radius of 134 ft (40.8 m) and being 13 ft (4 m) beyond the inner wall. Three feet (0.9 m) of insulating material is placed in the annular space beside the inner wall. Mass concrete fills the other 10 ft (3 m) providing resistance to external impact forces. Fig. 4a is a plan view showing the segments placed side by side. Fig. 4b shows a section of the double wall and the circumferential post-tensioning.

Figs. 4c and 4d show the tanks in various stages of construction.

Fig. 4c. Precast panel being readied into position on tank wall. Note connecting dowels on side of panel.

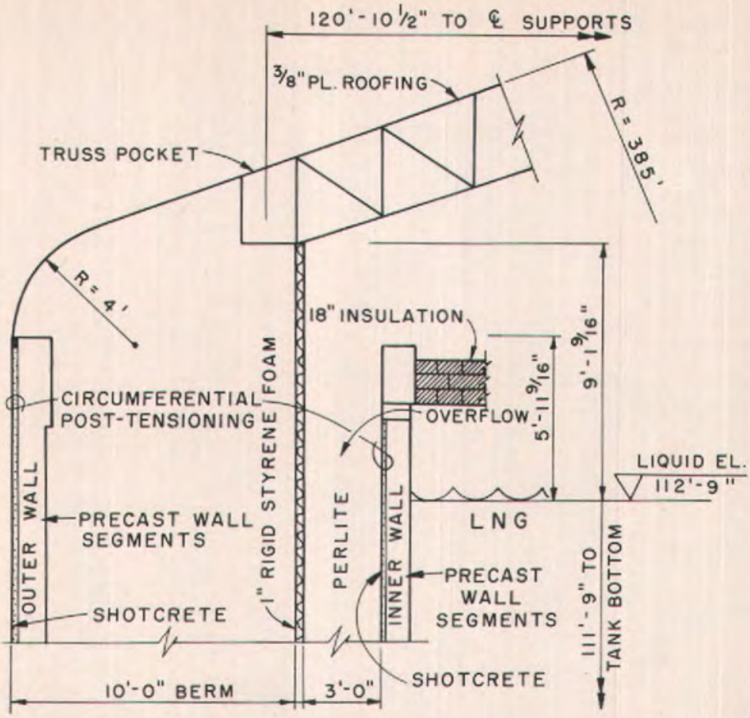



Fig. 4b. Section of double wall with post-tensioned precast wall segments.

Fig. 4d. Overall view of LNG tanks during construction.

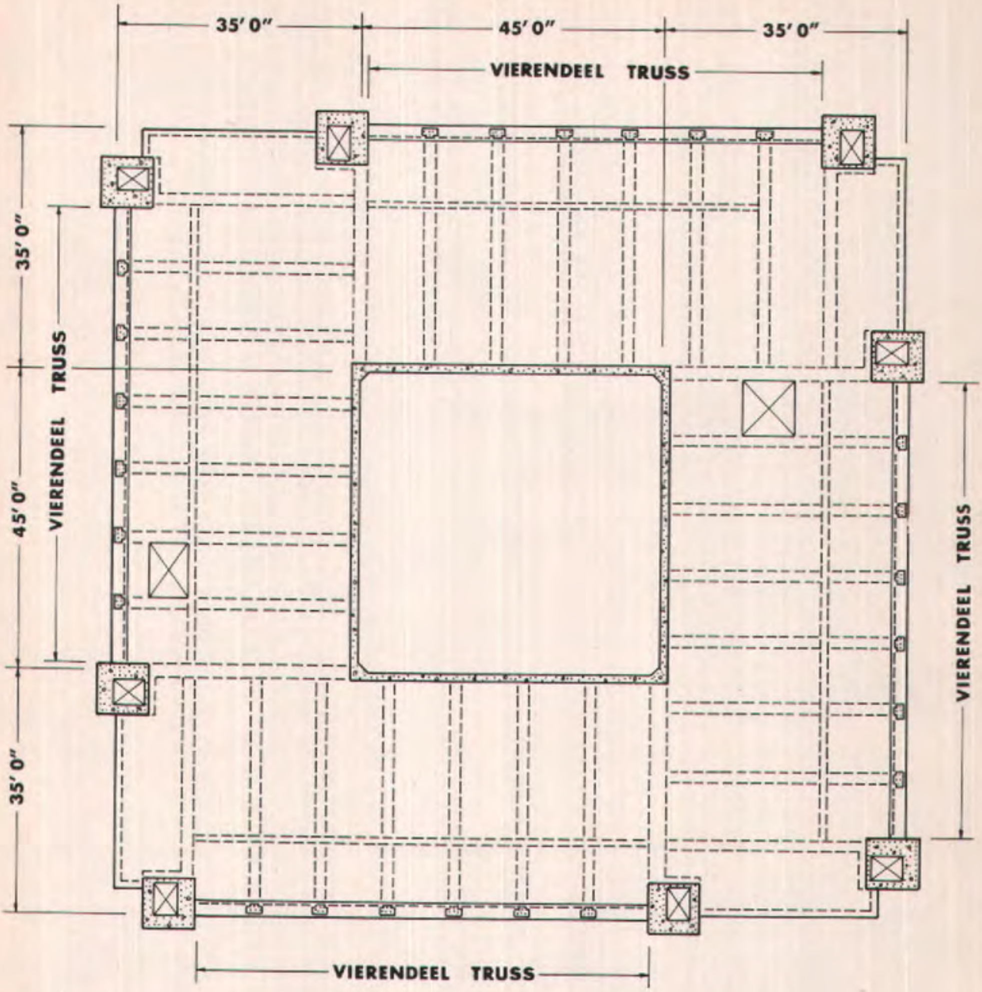



Fig. 5. Plan view of 28-story office building.

## 2.4 28-Story Office Building

In 1974, a 28-story office building[15] used a segmentally constructed Vierendeel truss as the exterior support for the edges of floors. Truss segments were block-I shaped, 17 ft (5.2 m) high, 10 ft (3 m) wide at top and bottom and weighed a maximum of 9 tons (8.2 t) each. The block-I's were erected side by side at the site and the top and bottom chords of the truss were post-tensioned horizontally. The ends of the truss were then anchored into large cast-in-place columns. Fig. 5 is a plan view showing the location of the trusses and columns. Fig. 6 is an elevation of a segmental Vierendeel truss. Further design and erection details of this building can be found in the Nov.-Dec. 1975 PCI JOURNAL.[15]

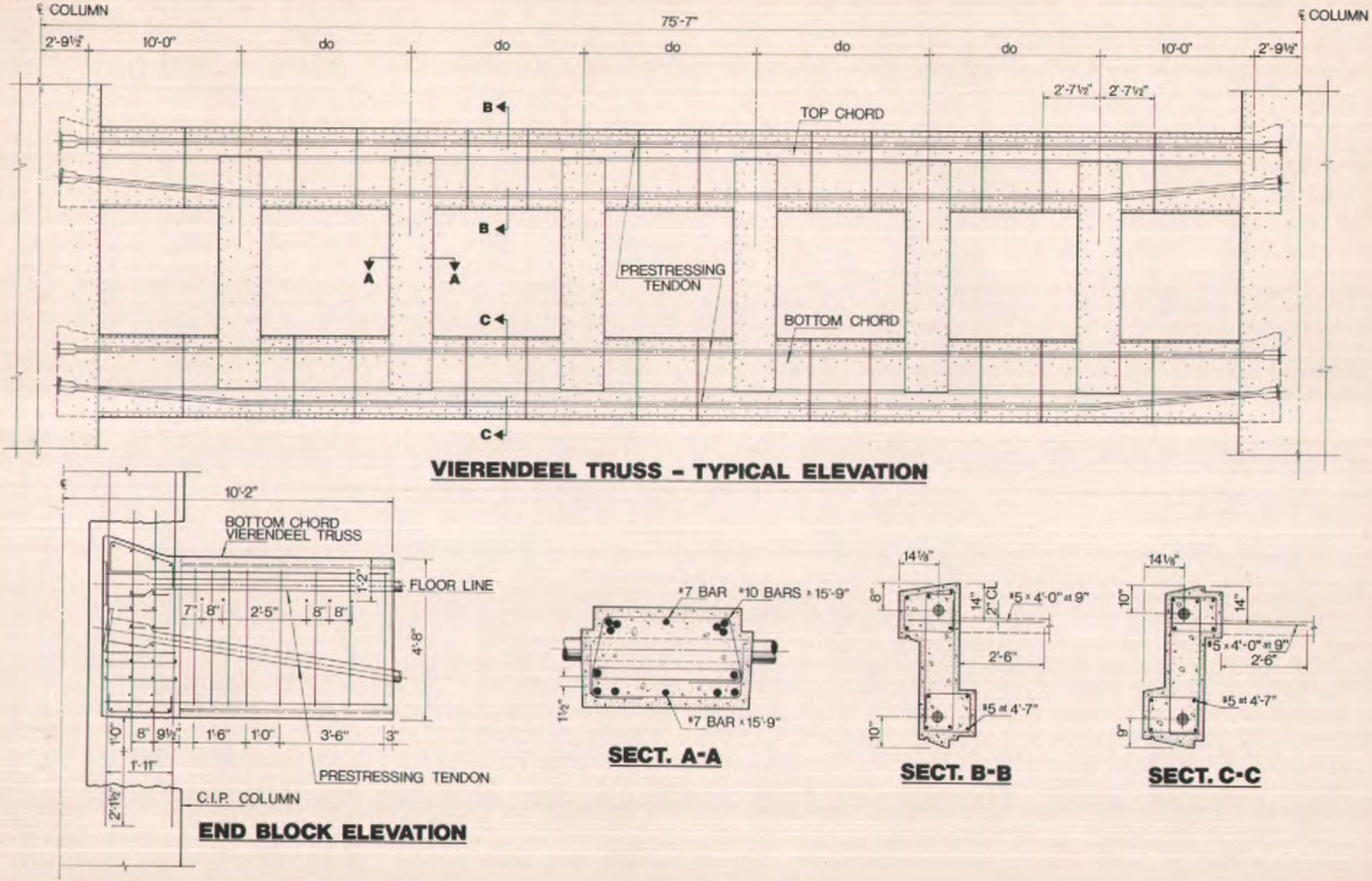



Fig. 6. Elevation of Vierendeel truss.

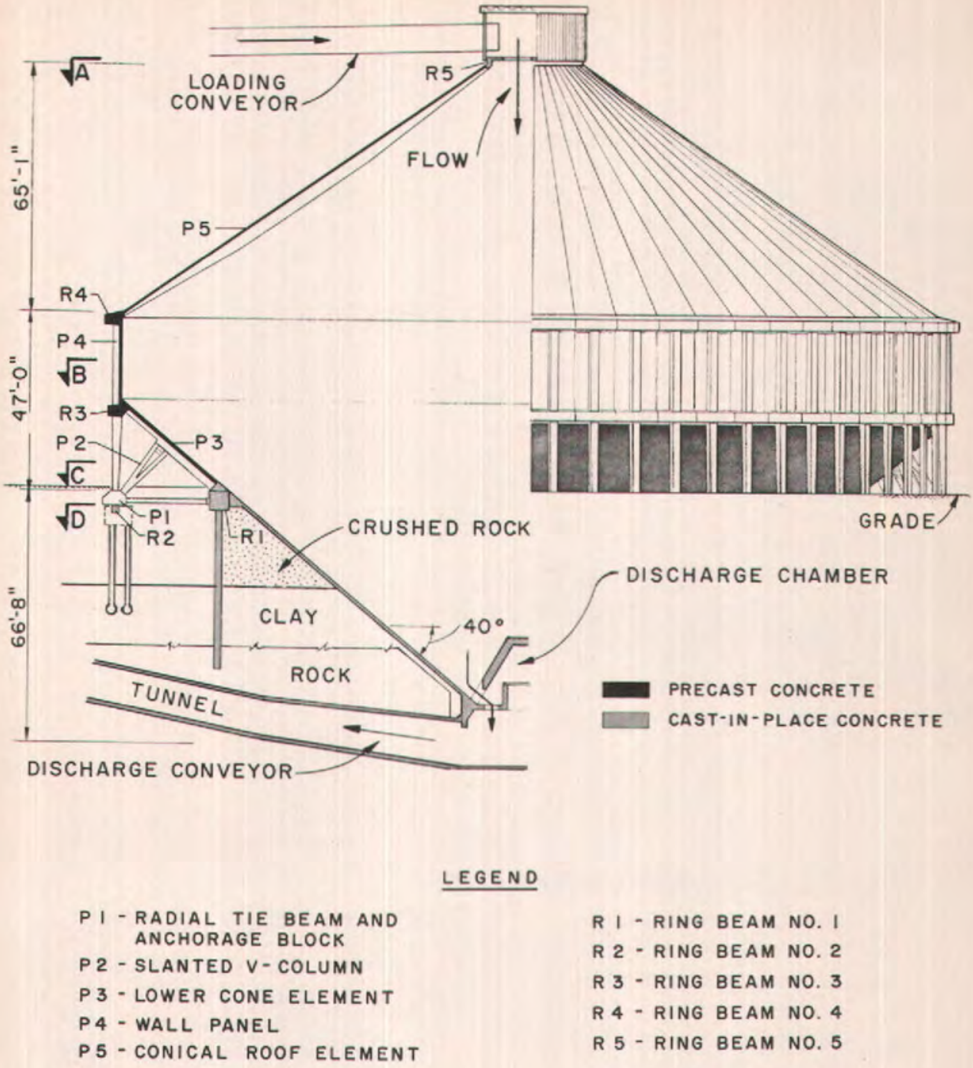



Fig. 7. Cross section of cement clinker silo showing location of main precast concrete components.

## 2.5 Cement Clinker Silo

A cement clinker storage silo[16] was built in 1975. It has a diameter of 214 ft (65.2 m), a height of 130 ft (39.6 m) above ground, and a depth of 79 ft (24.1 m) below grade. Three basic elements were composed of precast concrete: (a) a conical shell roof, (b) a segmental cylindrical wall which contained two levels of post-tensioned rings, and (c) an inverted cone bottom, two-thirds of which was below grade. Fig. 7 is a cross section of the silo showing its various components. Fig. 8 shows the individual precast components.

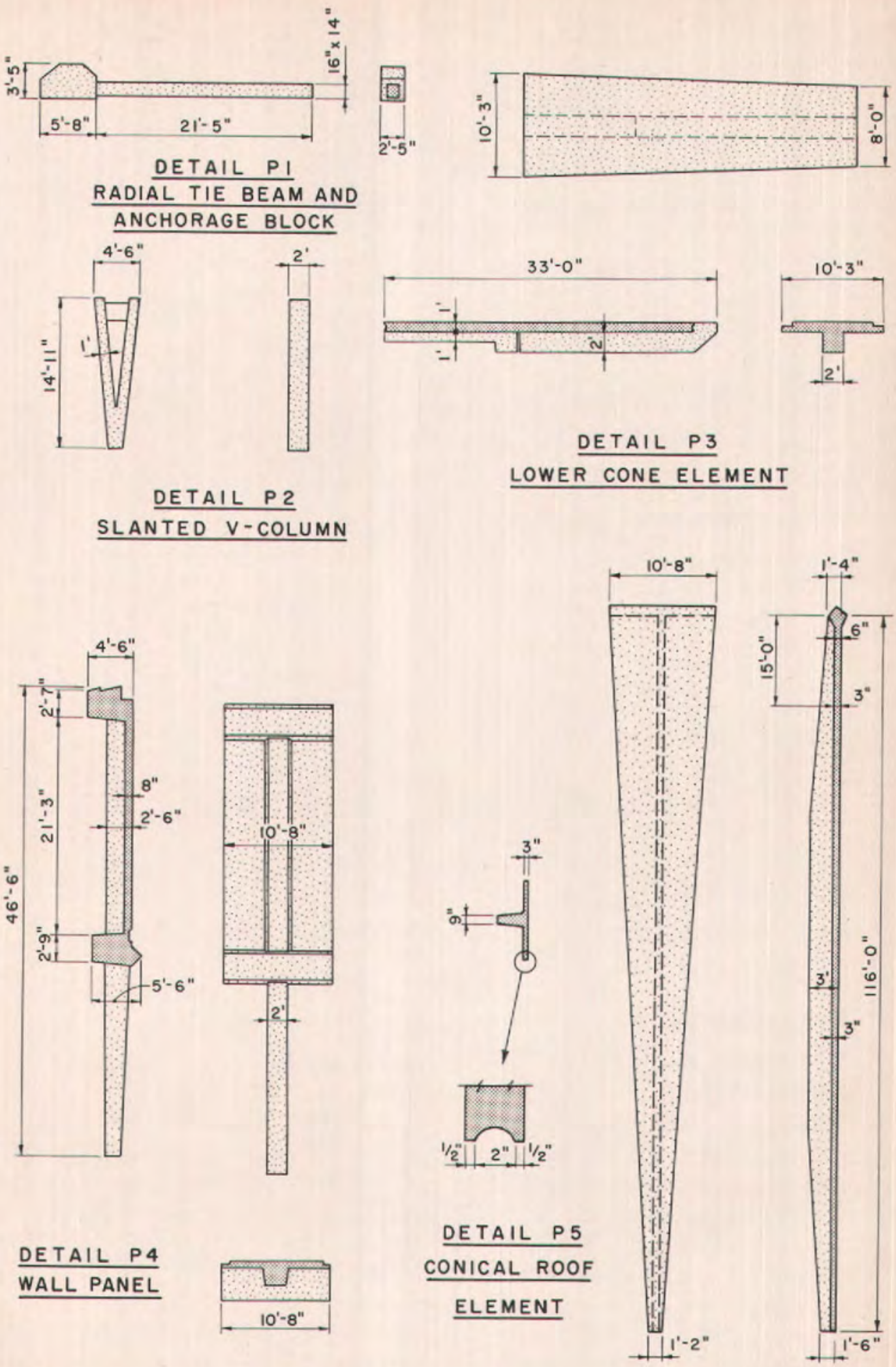



Fig. 8. Dimensional details of major precast concrete components.

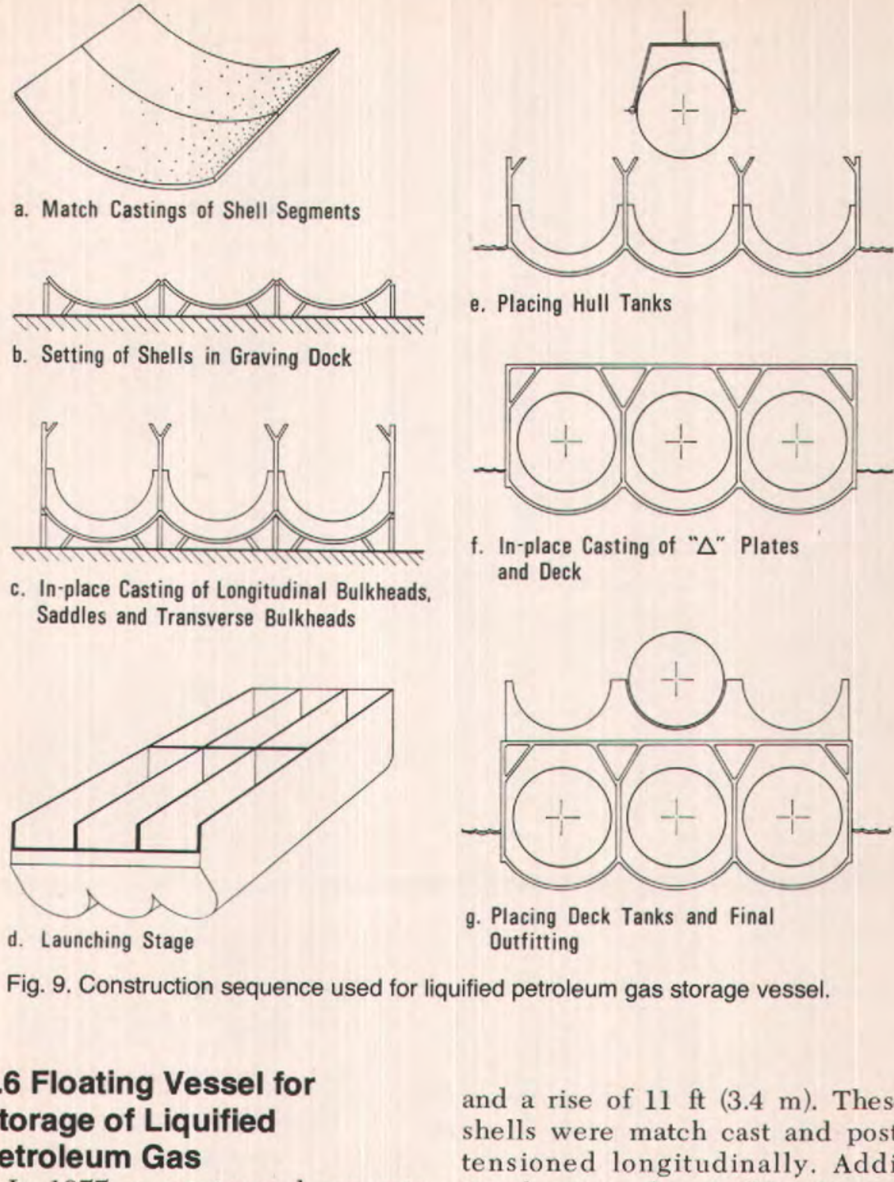



Fig. 9. Construction sequence used for liquified petroleum gas storage vessel.

## 2.6 Floating Vessel for Storage of Liquified Petroleum Gas

In 1977, a prestressed concrete floating vessel[17] for liquified petroleum gas was completed. The hull dimensions were 461 ft (140.5 m) long by 136 ft (41.5 m) wide. The hull bottom is in the shape of three cylindrical barrel shells. Each barrel consisted of precast curved segmental shells approximately 11 ft (3.4 m) long, 45 ft (13.7 m) wide and a rise of 11 ft (3.4 m). These shells were match cast and post-tensioned longitudinally. Additional cast-in-place bulkheads, saddles, stiffeners and decks resulted in the hull functioning as a multicell box girder and providing space and support for twelve 400-ton (363 t) steel tanks. Fig. 9 shows the construction sequence. Figs. 10, 11, and 12 are photographs of a hull segment being cast, delivered and erected.

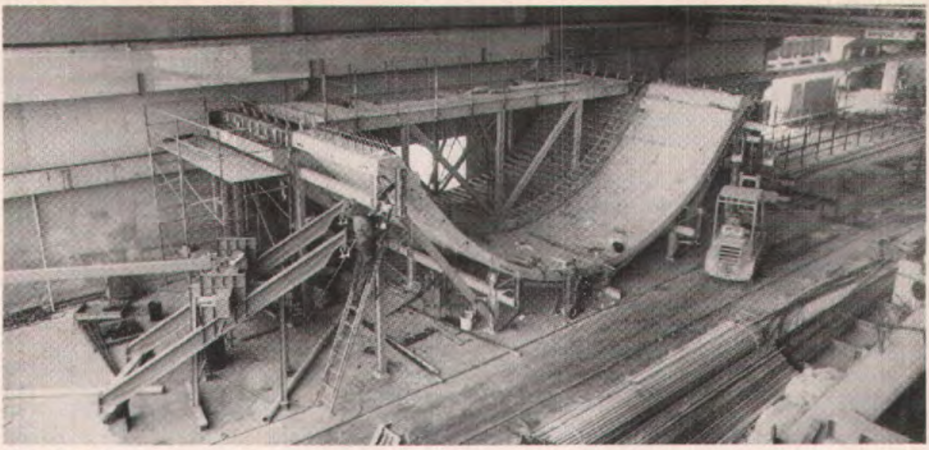


Fig. 10. Match-casting, precast concrete bottom shell segments.

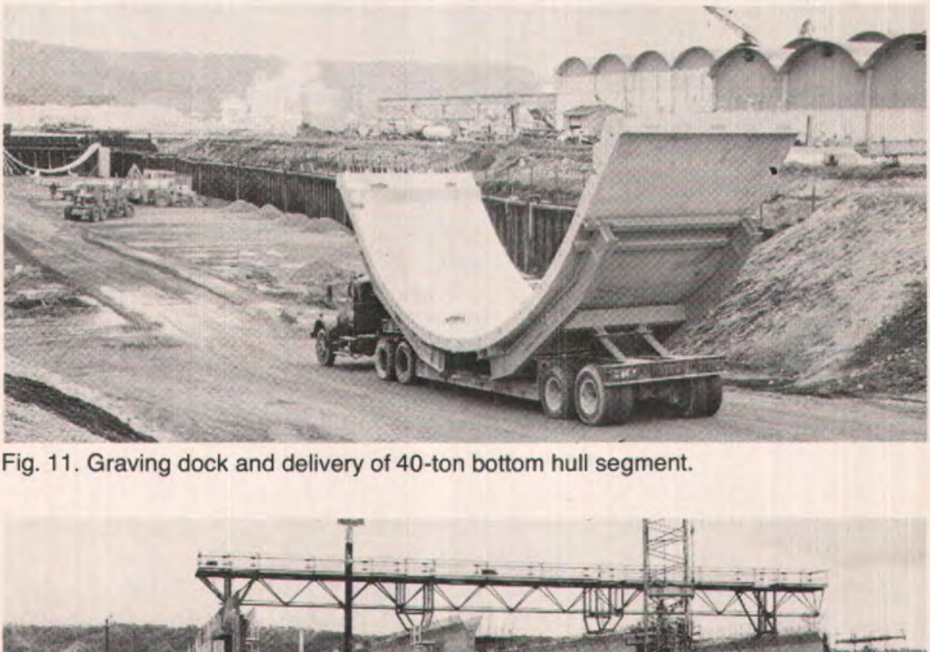


Fig. 11. Graving dock and delivery of 40-ton bottom hull segment.

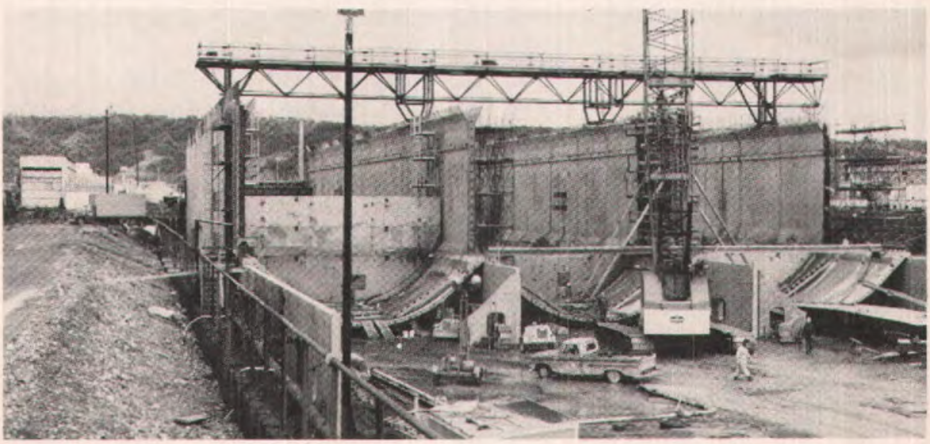


Fig. 12. Erection of the bottom shell.

Fig. 13. Five Points Station, Atlanta Rapid Transit System, Atlanta, Georgia.

## 2.7 Five Points Station, MARTA Rapid Transit, Atlanta, Georgia

The massive roof structure for this $42 million transit station (see Fig. 13) is made up entirely of precast components (some 450 in all) except for the reinforced concrete columns. In plan the roof is 167 ft (50.9 m) wide and 262 ft (79.86 m) long. It rises almost 50 ft (15.2 m) above the at-grade plaza level. The major elements of the roof structure are nine longitudinal beams 262 ft (79.86 m) long and eleven transverse beams, 167 ft (50.9 m) long. These beams are composed of up to 20 precast segments, 13 ft (4 m) long, 10 ft 8 in. (3.15 m) high and 2 ft 6 in. (0.76 m) wide. The main beams are match-cast construction glued together with epoxy. The heaviest precast pieces, weighing up to 23 tons (20.9 t) apiece, are those that make up the main beams.

## 2.8 Precast Segmental Bridge Construction

In precast segmental bridge construction, short segments are cast in a plant or near the job site under factory simulated conditions, but always at some location other than their final position in the structure, and then assembled in place. Precast segmental bridges may be classified by their method of construction.

Generally, the method of construction can be divided into four types:[18,19] (1) balanced cantilever, (2) span-by-span, (3) progressive placing, and (4) incremental launching or push-out construction.

### 2.8.1 Balanced Cantilever

In this method segments are cantilevered from a pier in a balanced fashion on each side until midspan is reached and a closure pour is made with a previous half-span cantilever from the preceding pier as shown in Fig. 14. This procedure is then repeated until the structure is completed.

Each segment added is immediately post-tensioned to the already completed portion of the structure. Care must be exercised to maintain compression over the entire joint area by means of temporary or permanent post-tensioning until all of the permanent tendons are stressed. The magnitude of stresses, segment alignment, and elevations must be evaluated at all stages of construction.

The advantages of cantilever construction are minimal interference with the environment, traffic flow is not obstructed below, small labor force required, and speed of erection.

An interesting concept in balanced cantilever segmental construction is its incorporation with cable-stayed bridges. This concept has been used in the United States for the Pasco-Kennewick Bridge[6-8] in the State of Washington as shown in Fig. 15.

### 2.8.2 Span-by-Span

This type of construction starts at one end of the structure and proceeds continuously to the other end of the structure, span-by-span, as opposed to the balanced cantilever where construction starts at the piers and symmetrically moves out from the piers. In this type of construction segments are supported during construction by a moveable falsework. The Long Key Bridge in

Fig. 14. Balanced cantilever construction employing a launching girder at the Sallingsund Bridge, Denmark.

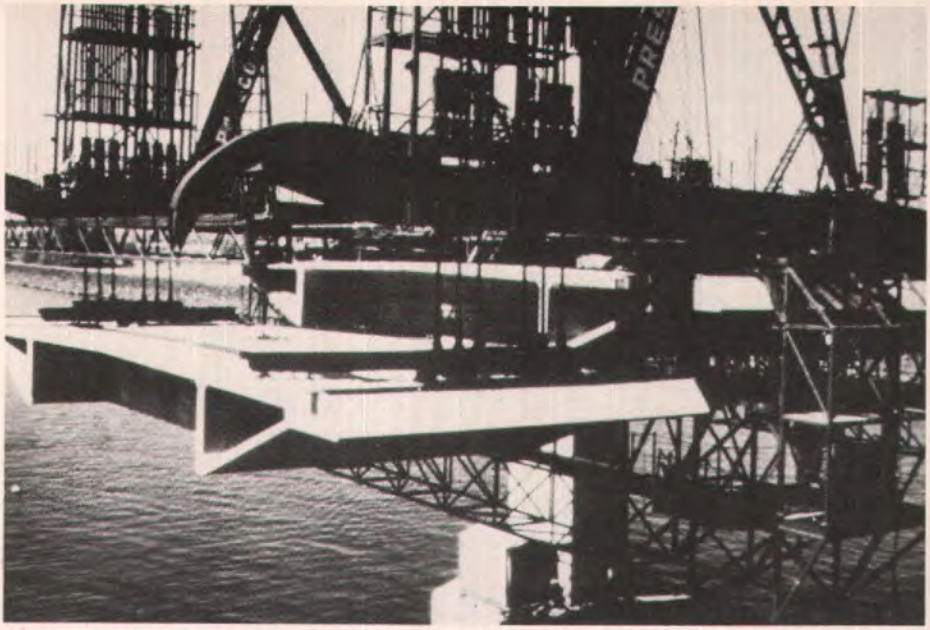


Fig. 15. Precast segments for balanced cantilever construction of the Pasco-Kennewick Bridge in Washington State.

Florida [see Fig. 16(a)] uses a structural steel trusswork soffit mounted on adjacent piers, which is moved by a barge-mounted crane from span to span.[20,21]

The segments could be assembled in final position on falsework (or fill), they could be assembled on the ground and lifted into place, or they could be assembled on falsework parallel to the final location and pushed laterally into place. The segments could also be partially assembled on the ground, lifted onto falsework, and the assembly completed on the falsework. An experimental bridge constructed at the Pennsylvania Transportation Research Facility[19] was erected on non-movable falsework as shown in Fig. 16(b).

**2.8.3 Progressive Placing**

Progressive placing is similar to the span-by-span method described above in that construction starts at one end of the structure and proceeds continuously to the other end of the structure. The progressive placing method is based on the balanced cantilever concept. In this method, the precast segments are placed continuously from one end of the structure to the other in successive cantilevers on the same side of the various piers, rather than by balanced cantilevers on each side of the pier.

Because of the length of cantilever (one span) in relation to construction depth, the erection stresses become prohibitive and temporary supports such as cable stays or pier bents are usually required. The erection procedure utilizing temporary stays is illustrated in Fig. 17. The first few seg-

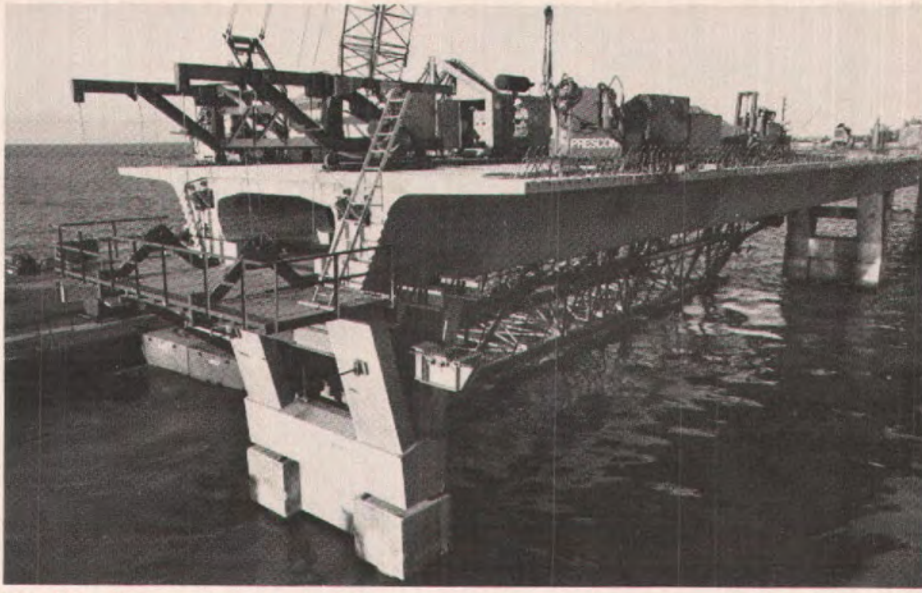


Fig. 16(a). Span-by-span technique using movable steel truss (Long Key Bridge).

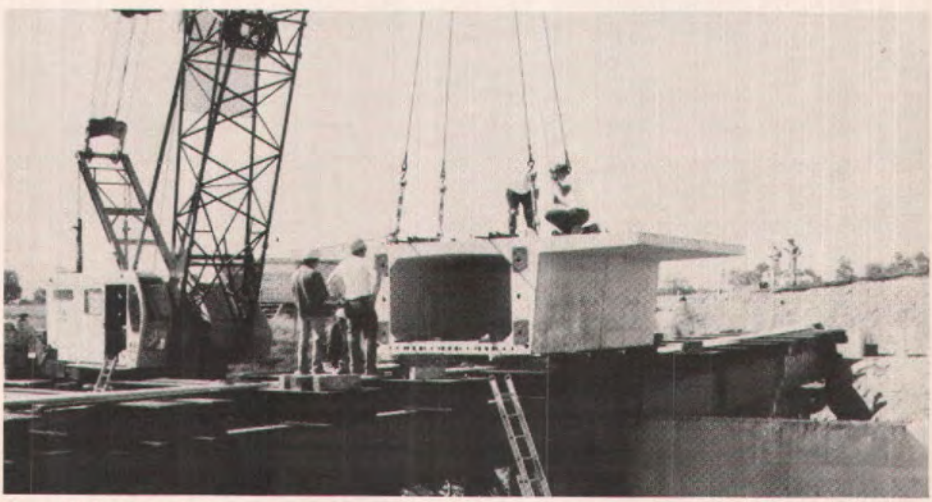


Fig. 16(b). Span-by-span technique using falsework at Pennsylvania Transportation Research Facility.

ments placed near the pier may be placed in a cantilever fashion until the stresses become excessive. At this point temporary stays (or some other temporary support) are required to control the cantilever stress.

This method can be used to advantage where site conditions prohibit the use of the span-by-span

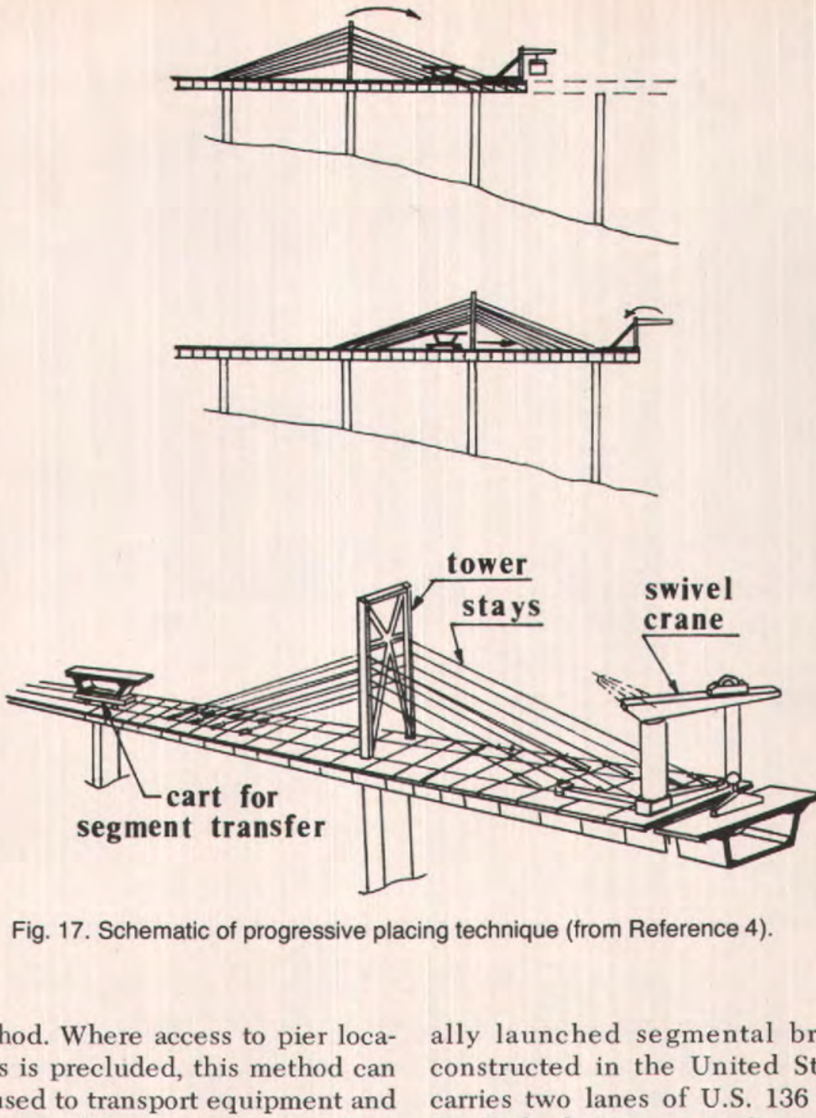



Fig. 17. Schematic of progressive placing technique (from Reference 4).

method. Where access to pier locations is precluded, this method can be used to transport equipment and materials for pier construction from above, over that portion of the completed structure. This method is of interest for applications in the 100 to 160-ft (30.5 to 48.8 m) span ranges.

### 2.8.4 Incremental Launching

Another variant of the segmental concept has evolved in Europe. In the United States, it is referred to as incremental launching or push-out construction. The first incrementally launched segmental bridge constructed in the United States, carries two lanes of U.S. 136 over the Wabash River near Covington, Indiana. It is a six-span structure with end spans of 93 ft 6 in. (28.5 m) and four interior spans of 187 ft (57 m).[22,23]

In this type of construction segments of the bridge superstructure are positioned to the rear of the abutment and post-tensioned together. The assembly of units is launched stepwise forward one unit at a time to allow for succeeding

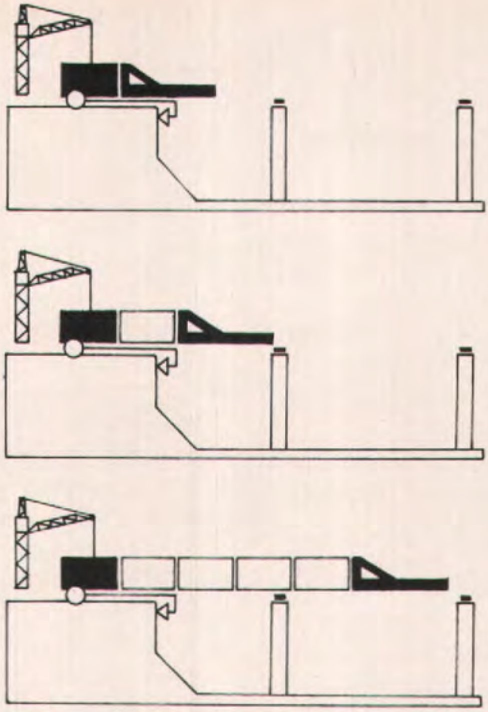


Fig. 18. Schematic of incremental launching sequence (Courtesy of F. Leonhardt).

units to be positioned to the rear of the abutment.

Bridge alignment in this type of construction may be either straight or on a curve; however, the curve must be a constant radius curve, vertically and/or horizontally.

Longitudinal prestressing consists of two families of tendons: tendons concentrically placed and tensioned before launching, and tendons placed and tensioned after launching, that is, negative moment tendons over the supports and positive moment tendons in the bottom of the section in the central portion of the span.

Hydraulic jacks are used to progressively launch successive concentrically prestressed units in a longitudinal direction as shown in Fig. 18. To allow the superstructure to move forward, special sliding bearings are provided. To reduce the large negative bending moments that the structure would be subjected to during launching, a fabricated structural steel launching nose is attached to the lead segment. Long spans are subdivided by means of temporary piers to keep negative moments at an acceptable level.

Complete coordination of all operations is essential to prevent the occurrence of a "runaway structure," unexpected twist, or excessive displacement of the structure while on or between temporary or final supports.

# CHAPTER 3 — FABRICATION OF PRECAST SEGMENTS

## 3.1 General Considerations

It is generally preferable to use as few units as possible, consistent with economic shipping and erection. During design of a segmental structure, consideration should be given to the selection of cross sections which simplify formwork, lead to production efficiency, and overall project economy.

In the case of girder segments, economy and speed of production may be increased by:

(a) Keeping the length of the segments equal and keeping them straight, even for curved structures.

(b) Proportioning the segments or parts of them, such as keys, in such a way that easy stripping of the forms is possible.

(c) Maintaining a constant web thickness in the longitudinal direction.

(d) Maintaining a constant thickness of the top flange in the longitudinal direction.

(e) Keeping the dimensions of the fillet between webs and the top flange constant.

(f) In case of variable depth, keeping webs vertical to avoid a variable bottom flange width.

(g) Avoiding interruptions of the surfaces of webs and flanges caused by protruding parts for anchorages, inserts, etc.

(h) Using a repetitive pattern, if practical, for tendon and anchorage locations.

(i) Minimizing the number of diaphragms and stiffeners.

(j) Avoiding dowels which pass through the forms, when possible.

Because of economic considerations the variation of the cross section of girder segments is generally limited to changing the depth and the thickness of the bottom flange. Curves in the vertical and horizontal direction and superelevation of the structure can be easily accommodated.

If a segment is damaged beyond repair in transportation or erection, a new segment should be cast. It is preferable to match-cast it against one or both of the mates but, if not possible, a tolerance provision should be made for erecting the new segment which will not adversely affect the geometry or aesthetics of the completed structure. If damage occurs to the joint area during transporting or erection, the segment may be erected at the discretion of the Engineer. Generally, this will be allowed when the damage is to a small area and the damage is patched after erection. This precludes destroying the fit of the match-cast surfaces. The location of the damage and its effect on the segment's strength and serviceability should be considered in the Engineer's decision.

Evaluation of segments having manufacturing defects should recognize that many of the segments in a segmental bridge may have excess capacity since concrete dimensions are kept constant for standardization purposes and controlling sections may be located elsewhere in the structure. Bottom slab and web dimensions, for example, are usually critical only in the support area. Honeycombing of elements outside this area, provided it is suitably repaired, need not be cause for rejection.

## 3.2 Methods of Casting

Segments to be erected with wide joints may be cast separately. Match-cast joint members are cast by the "long-line" or "short-line" method.

### 3.2.1 The Long-Line Method

*Principle* — All of the segments are cast, in correct relative position, on a long line. One or more formwork units move along this line. The formwork units are guided by a pre-adjusted soffit. An example of this method is shown in Figs. 19a to 19c. For further practical details of the long-line fabrication method see Reference 24.

*Advantages* — A long line is easy to set up and to maintain control over the production of the segments. After stripping the forms it is not necessary to take away the segments immediately.

*Disadvantages* — Substantial space may be required for the long line. The minimum length is normally slightly more than half the length of the longest span of the structure. It should be constructed on a firm foundation which will not settle or deflect under the weight of the segments. Because the forms are mobile, equipment for casting, curing, etc., has to move from place to place. In case the structure is curved, the long line must be designed to accommodate the curvature. This, however, increases the complexity of the form and the more appropriate method to produce curved bridges is by the short-line method.

### 3.2.2 The Short-Line Method

*Principle* — The segments are cast at the same place in stationary forms and against a neighboring element. After casting, the neighboring element is taken away and the last element shifted to the place of the neighboring element, clearing the space to cast the next element. A horizontal casting operation is illustrated in Figs. 20a through 20c. Segments intended to be used horizontally may also be cast vertically.

*Advantages* — The space needed for the short-line method is small in comparison to the long-line method, approximately three times the length of a segment. The entire process is centralized. Horizontal and vertical curves and twisting of the structure are economically obtained by adjusting the position of the neighboring segment.

*Disadvantages* — To obtain the desired structural configuration very precise workmanship, quality control procedures, and geometry control are required. The neighboring segments must be accurately positioned for each match-casting.

## 3.3 Formwork

### 3.3.1 General

All forms should be carefully designed and it is highly recommended that forms be designed by Engineers with experience in the segmental method and the procedure, as well as form design. Formwork should be designed to safely support all loads that might be applied without undesired deformations or settlements. Soil stabilization of the foundations may be required, or the formwork may be designed so that adjustments can be

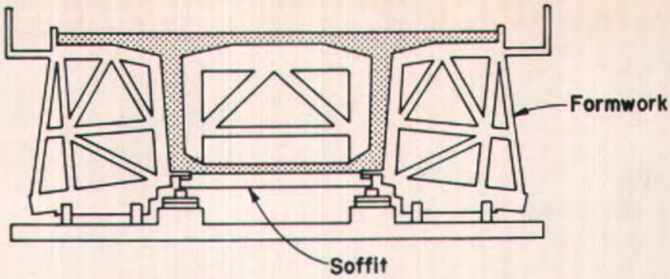



Fig. 19(a). Cross section of formwork using long-line method.

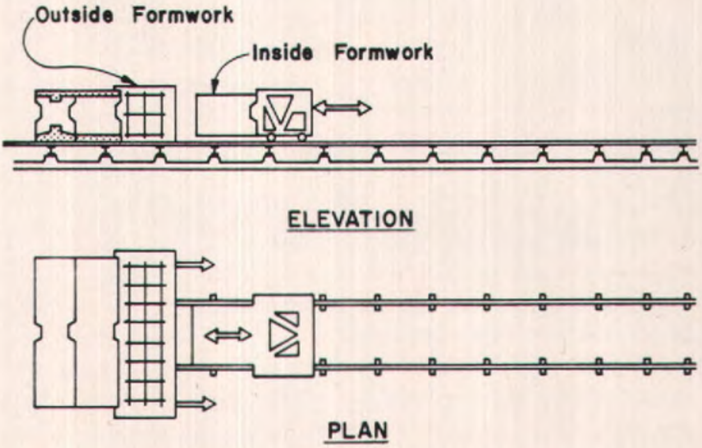



Fig. 19(b). Formwork at start of casting (long-line method).

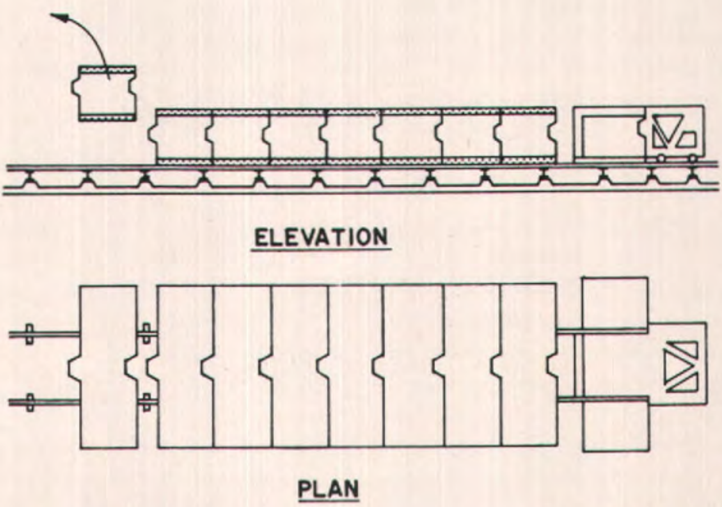



Fig. 19(c). Formwork after casting several segments (long-line method).

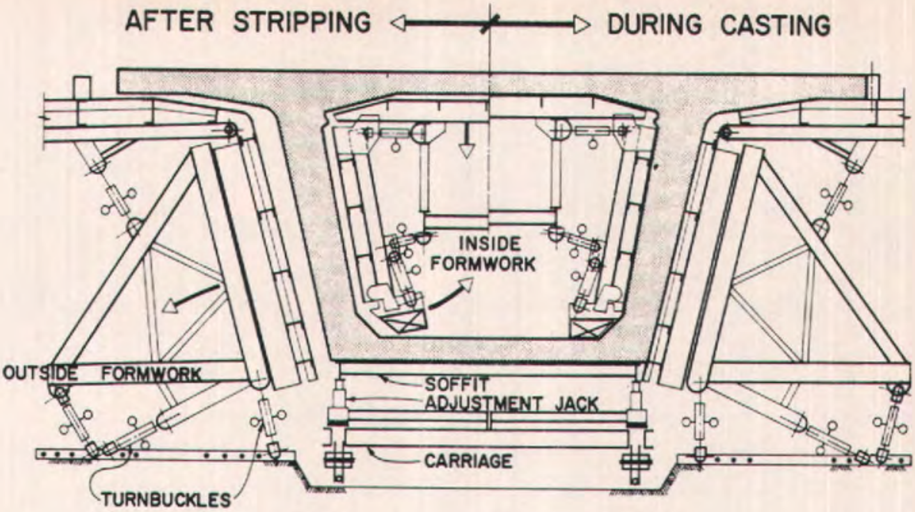



Fig. 20(a). Formwork for short-line method. It should be designed to facilitate inspection.

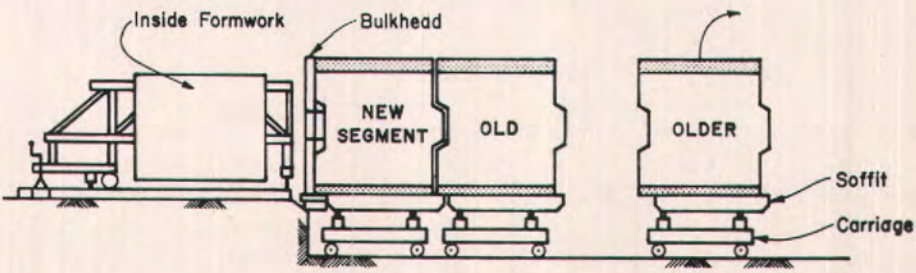



Fig. 20(b). Formwork just before separation of segments (short-line method).

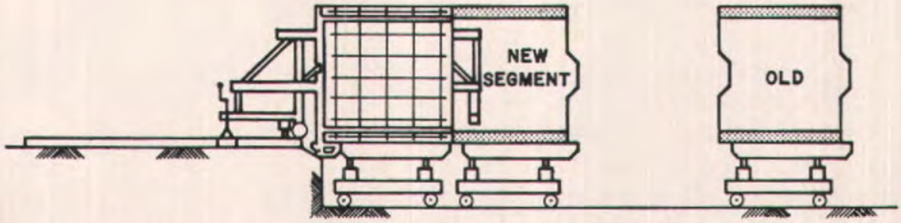



Fig. 20(c). Formwork just before casting next segment (short-line method).

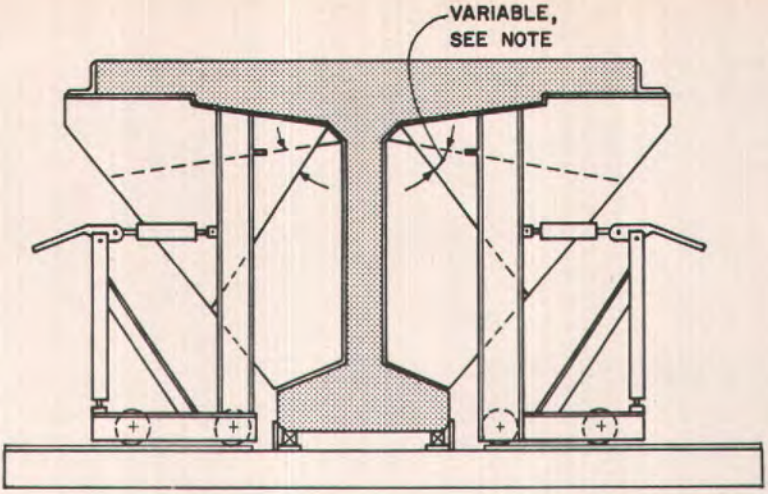



Fig. 21. Form used to construct I-shaped girder segments. (Note: Plane of deck may be varied with respect to web of I-beam).

made to compensate for settlement.

Since production of segments is based on multiple reuse of forms, the formwork should be sturdy and special attention should be given to construction details. Forms should also be easy to strip and handle. An example of a form used to construct I-shaped girder segments is shown in Fig. 21.

Cement paste leakage through formwork joints should be prevented. This can normally be achieved by using a flexible sealing material. Special attention should be given to the junction of tendon sheathing with the forms.

The forms may need to be flexible in order to accommodate slight differences of dimensions with the previously cast segment. They should be designed in such a manner that the necessary adjustments for the desired camber, curvature and twisting can be achieved accurately and easily.

Special consideration should be given to those parts of the forms that have to change in dimensions. To facilitate alignment or adjustment, special equipment, such as screw or hydraulic jacks, should be provided.

Attachments to the form such as anchorages, sheathing, blockouts and inserts should be designed in such a way that their position is rigid during casting. Fittings should not interfere with stripping of the forms.

Accelerated curing may cause temperature gradients in the concrete which may be harmful to the segments especially in the cooling phase. Cantilever flanges and other thin elements cool faster than heavier parts of the section and cracking of elements has occurred due to this phenomenon. This should be guarded against by close control of the heating and cooling process.

When using accelerated curing it is furthermore advisable to cure the segment being cast together with the segment it was cast against. This prevents temperature deformations of the hardened segment during curing of the new segment which may lead to open joints upon erection.

Internal vibration is commonly used. If the form is designed for external vibrators they should be attached at locations that will achieve optimum consolidation and permit easy replacement during the casting operations. To break the suction during stripping, air jets may be employed if required.

#### 3.3.2 Provisions for Tendons

Holes for prestressing tendons may be formed by:

(a) Sheathing which remains after hardening of the concrete. Flexible sheathing made of spirally wound metal may be stiffened from the inside by means of inflatable rubber duct tubes during the casting operation.

(b) Rigid sheathing with smooth or corrugated walls may be used that will not deform excessively under the pressure of vibrating the plastic concrete. Sheathing should be carefully aligned and tied at close intervals to avoid displacement.

Holes should be accurately positioned, particularly when a large number of holes are required. Often, sheathing with tendons in place require no stiffening. Further information on forming holes for tendons and tendon anchorages is presented in Chapter 6.

#### 3.3.3 Provisions for Supplemental Reinforcement

Supplemental mild steel reinforcement is used to resist forces not provided for by prestressing. Chapter 7 lists the requirements for supplemental reinforcement.

#### 3.3.4 Tolerances

Tolerances as specified by applicable codes of practice can generally be used for segmental construction. It should be borne in mind, however, that tolerances on dimensions which directly influence alignment or shape of the erected segmental structure require particular attention. Formwork for segmental box girder bridge construction that meet the tolerances shown in Table 1 (see next page) are normally considered acceptable.

Depending upon the detail at bridge piers, the tolerances for the soffit of the pier segment may need special attention. The actual dimensions of a segment should be determined immediately after removing the forms. If specified tolerances are exceeded, acceptance or rejection should be based on the effect on final alignment and on whether the effect can be corrected in later segments.

### 3.4 Concrete

Uniform quality of concrete is essential for segmental construction. Procedures for obtaining high quality concrete are covered in ACI,[25] PCI,[26] and PCA[27] publications.

Both normal weight and structural lightweight concrete can be made consistent and uniform with proper mix proportioning and production controls. Normally concrete for segmental construction will have a slump of 3 to 5 in. (76 to 127 mm) and 28-day strength greater than the strength specified by structural design. It is recom-

**Table 1. Formwork tolerances for segmental box girder bridge construction. (To correlate tolerances, see sketches on next page.)**

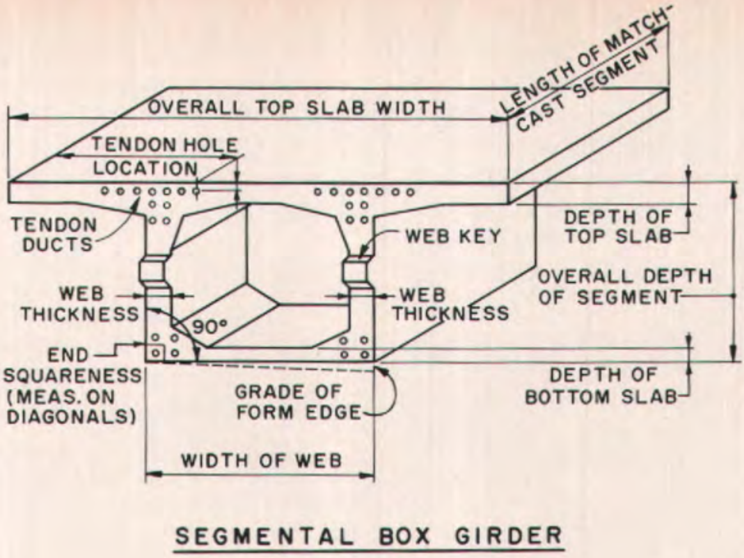
| | |
|---|---|
| Web thickness | ± ¼ in. (6.4 mm) |
| Depth of bottom slab | ± ¼ to 0 in. (6.4 to 0 mm) |
| Depth of top slab | ± ⅛ in. (3.2 mm) |
| Overall depth of segment | ± 1/32 in./ft (2.6 mm/m), of depth, ± ½ in. max. (12.5 mm) |
| Overall top slab width | ± 1/32 in./ft (2.6 mm/m), ± 1 in. max. (25 mm) |
| Length of match-cast segment (not cumulative) | ± ⅛ in./ft (10.4 mm/m), +1 in. max. (25 mm) |
| Diaphragm thickness | ± ¼ in. (6.4 mm) |
| Grade of form edge and soffit | ± 1/16 in. in 10 ft (0.5 mm/m) |
| Tendon hole location in bulkhead | ± 1/16 in. (1.6 mm) |
| Position of shear keys | ± ⅛ in. (3.2 mm) |
| Vertical plumbness | ± 1/32 in. (0.8 mm) |
| End squareness | ± 1/32 in. (0.8 mm) |
| **Finished segment tolerances should not exceed the following:** | |
| Web thickness | ± ⅜ in. (9.5 mm) |
| Depth of bottom slab | + ½ to 0 in. (12.7 to 0 mm) |
| Depth of top slab | ± ¼ in. (6.3 mm) |
| Overall top slab width | ± 1/16 in./ft (5.2 mm/m), ± 1 in. max. (25 mm) |
| Diaphragm thickness | ± ½ in. (12.5 mm) |
| Grade of form edge and soffit | ± ⅛ in. in 10 ft (1.0 mm/m) |
| Tendon hole location | ± ⅛ in. (3.2 mm) |
| Position of shear keys | ± ¼ in. (6.3 mm) |
| Vertical plumbness | ± 1/16 in. (1.6 mm) |
| End squareness | ± 1/16 in. (1.6 mm) |

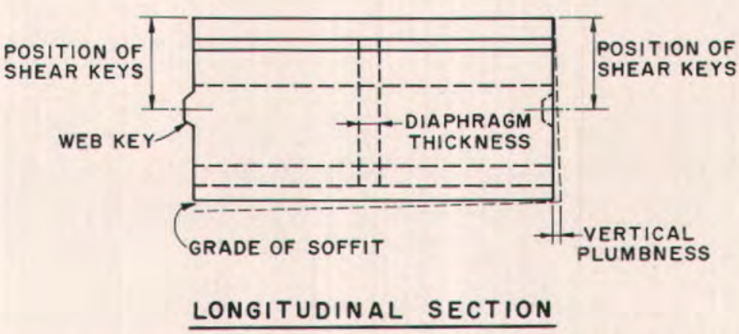
mended that statistical methods be used to evaluate concrete mixes.

The methods and procedures used to obtain the characteristics of concrete required by the design may vary somewhat depending on whether the segments are cast in the field or in a plant. The results will be affected by curing temperature and type of curing. Liquid or steam curing or electric heat curing may be used.

A sufficient number of trial mixes should be made to assure uniformity of strength and modulus of elasticity. Careful selection of aggregates, cement, admixtures and water will improve strength and modulus of elasticity and will also reduce shrinkage and creep. Soft aggregates and poor sands should be avoided. Creep and shrinkage data for concrete mixes should be available or should be determined by tests.

It is dangerous to use corrosive admixtures such as calcium chloride or those containing substantial chlorides, nitrates, sulphates and fluorides. Water-reducing admixtures and also air-entraining admixtures which improve concrete resistance to environmental effects, such as deicing salts and freeze and thaw actions, are highly desirable. However, their use should be rigidly controlled in order not to increase undesirable variations in strength and modulus of elasticity of the concrete.




Isometric view and longitudinal section of box girder showing tolerances designated in Table 1.

Mixes should be designed to minimize creep and shrinkage. Reliable data on the potential of the mix in terms of strength gain and creep and shrinkage performance should be used as a basis for improved design parameters.

Proper vibration should be used to afford use of lowest slump concrete and to allow for the optimum consolidation of the concrete.

Consideration should be given to delaying erection of segments until their internal moisture is reduced to a level that subsequent drying will not produce unexpected deformations.

## 3.5 Joint Surfaces

Requirements concerning surface condition should be stricter for match-cast joints than for wide joints filled with mortar or concrete.

### 3.5.1 Quality

Best results in match-casting are achieved if the joint surfaces are even and smooth. Surface crushing or chipping off of edges during post-tensioning caused by point contact are thus avoided.

Chamfers at match-cast joints are not recommended since they are difficult to manufacture and decrease the joint contact area considerably. In case chamfers are considered desirable, as may be expected in buildings or special structures with exposed "architectural" surfaces, the effect of the area reduction should be duly investigated. For wide joints, rough surfaces are preferable as they produce better bond between segment and filling material.

### 3.5.2 Holes for Tendons and Couplers

Holes or sheathing for tendons should be located very precisely when producing segments joined by post-tensioning. Care is required to prevent leakage or penetration of joint-filling materials into the duct so as to prevent blocking the passage of the tendons.

## 3.6 Bearing Areas

Bearing areas at reactions should be smooth and in full contact to assure uniform distribution of bearing forces. If possible, in view of manufacturing and erection tolerances, it may be desirable to place bearing elements like pads or steel plates in the forms before casting. Otherwise, cement mortar or an epoxy mortar may be required between the segment and bearing area to ensure full contact.

# CHAPTER 4 — TRANSPORTATION, STORAGE AND ERECTION OF PRECAST ELEMENTS

## 4.1 General

Due to the interdependence which exists between design, fabrication, and erection, close coordination is required and is of major importance. The design concept, by specifying member size, generally dictates the choice of transportation methods. Erection methods may also be dictated by the design concept.

## 4.2 Plant Handling and Transportation

Segments should be handled carefully, without impact, in a manner that limits stresses to values compatible with the strength and age of the concrete. Location of lifting hooks and inserts should be determined carefully to prevent damage of segments during handling. Lifting hooks or inserts should have an adequate safety factor, a minimum of 4.0 on handling loads plus 50 percent impact is suggested. Transportation over uneven surfaces may produce static and dynamic stresses which need to be considered, especially at an early age of a segment. Special care should be exercised during handling and transportation to protect cantilevers or projections against damage or cracking.

Plant handling and transportation greatly influence the construction cost. The way the segment is transported from the casting yard into its final position, the site conditions,

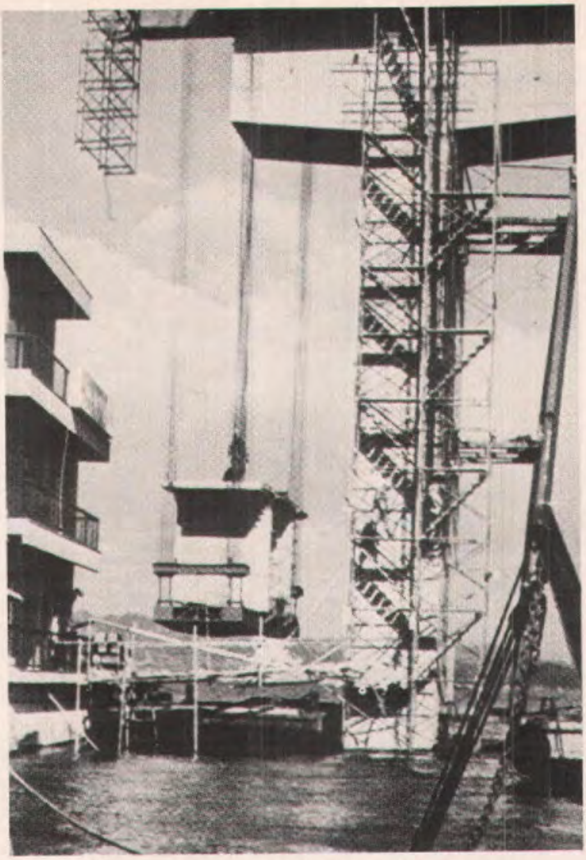


Fig. 22. Winch and beam erection method.

segment weight and number of rehandlings are major parameters influencing costs.

Since it is essential to prevent deformation of the segments caused by uneven settlement, especially twisting, storage should be on sound supports. Segments should not be unnecessarily exposed to uneven shrinkage caused by moisture loss due to excessive wind or partial sunlight exposure. Storage of segments should be arranged to minimize damage, deflection, twist, and discoloration of the segments. Stacking should be limited to avoid excessive direct or eccentric forces. Stacking may induce deformations in the segment which may "lock-in" because of creep and additional strength gain of segment concrete while in storage.

Inserts, anchorages, and other embedded items may need to be protected from corrosion, and from penetration of water or snow during cold weather. Contact surfaces to be bonded with epoxy should be kept free of contaminants.

Scheduling of segments is very important; fabrication, transportation, and erection schedules should be coordinated so that at least the next matching segment is delivered to the job site before the previous segment is erected.

Transportation may be by truck, rail, or barge or a combination of such modes. Modification of transportation vehicles or equipment may be dictated by size and weight of the segment. Local hauling regulations may limit width, length, height, and weight of segments for transportation over highways without special permits.

## 4.3 Erection Methods

Erection is commonly accomplished by truck, crawler, or barge mounted cranes; winch and hoist; launching girder; or at times, by specially designed equipment custom made for the project.

Several parameters will influence the method of erection. The size and weight of the unit will determine, or be determined by, the erection equipment available. The height of the structure above existing terrain, or navigation clearance, will determine if the units can be erected by truck, crawler, or barge mounted cranes. In some cases, precast units may be lifted up from a barge or truck by a winch and hoist located on the pier or completed portion of the structure. If the height of the structure is such that truck or barge cranes are not practical or if the structure is a long viaduct, a movable launching girder or a movable falsework may be necessary and economically justifiable.

### 4.3.1 Cranes

Mobile cranes moving on land or floating on barges may be used where access is easily available. Portal cranes straddling or attached to the decks have also been used. Cranes can be used with balanced cantilever, span-by-span, and progressive placing types of construction.

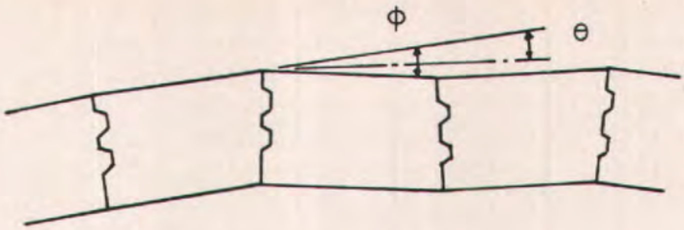
#### 4.3.2 Winch and Beam

The winch and beam (see Fig. 22) is a lifting device that extends over the completed portion of the bridge deck on a short cantilevered mechanism anchored to the bridge deck. From this position, a segment is lifted and held in place until post-tensioning can occur. After post-tensioning, the winch and beam is moved onto the newly added segment and the process is repeated.

Precast column segments may be placed on the column with the same basic equipment cantilevered temporarily out from a tower attached to the completed portion of the column. The winch and beam is commonly used with balanced cantilever and progressive placing types of construction.

#### 4.3.3 Launching Girder

The launching girder (see Fig. 14) is a special mechanism that travels along the completed deck spans and maintains the work flow at that level. The essential parts of the launching girder are a main truss with a length somewhat greater than the maximum bridge span, three leg frames which are attached to the main truss, and a trolley which travels along the bottom chord of the girder and is capable of moving the segment in the longitudinal, transverse, and vertical directions. During construction, by assuming various positions, the girder can place seg-




$\phi$ = Real slope change

$\theta$ = Theoretical slope change

$$|\phi - \theta| \leq 0.3\%$$

Fig. 23. Longitudinal slope change tolerance.

ments in cantilever, place segments over piers, and can move to the next span so that the construction process can be repeated. This is the fastest method of cantilever construction, but it is limited to large projects because of the high initial cost of the launching girder.

## 4.4 Erection Considerations

Segments should be stabilized and braced to resist construction loads, impact, wind loads, accidental forces, etc. Segments must be free from restraint during post-tensioning.

Depending on the type of construction/erection methods utilized, changes in post-tensioning forces may affect various conditions which should be considered in design. These may be the redistribution of forces at supports, stability due to prestressing, effect of reactions (shear) at temporary supports or the additional loads imposed by heavy handling, lifting equipment or devices.

The shortening of the segments and jointing material due to previous post-tensioning, temperature effects, settlement or other loading conditions should be investigated before final post-tensioning of the structure.

## 4.5 Erection Tolerances

The erection tolerances discussed below apply principally to segmental box girder bridges.

Maximum deviation between outside faces of adjacent segments in the erected position should not exceed ¼ in. (6 mm). Longitudinally, the angular deviation from the theoretical slope change between two successive segments is recommended not to exceed 0.3 percent as shown in Fig. 23. Transversely, the angular deviation from the theoretical slope difference between two successive segment joints is recommended not to exceed 0.1 percent as shown in Fig. 24.

The most important item of tolerance or acceptance is the final overall geometry of the erected superstructure. The elevation of the deck surface in the completed structure should not vary substan-

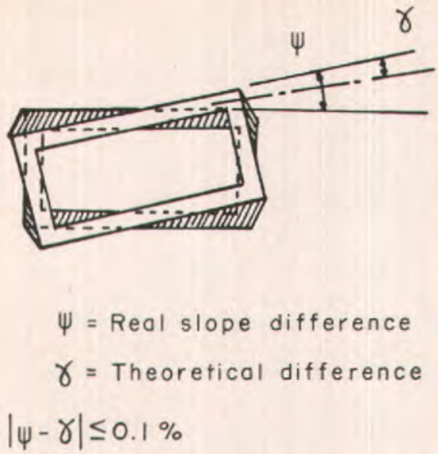



Fig. 24. Transverse slope difference tolerance.

tially from the theoretical profile grade elevation. In cantilever built structures the final elevation is obtained only after completion of the closure sections between successive cantilevers, subsequent post-tensioning, and after all creep and shrinkage have taken place.

For bridge decks more liberal tolerances may be accepted if the design incorporates a wearing surface and/or separately cast traffic barriers or edge beams which permit a smoothing out of small alignment and elevation errors. For this reason, it is recommended that traffic barriers and edge beams be cast separately (they may be cast in place or precast) and not precast with the segments.[28]

In case correction in the alignment is required, adjustments should be acceptable to the Engineer. Stainless steel wire mesh 1/16 in. (1.6 mm) thick with mesh dimensions of 1/8 in. (3.2 mm) embedded in the epoxy joint have been used for this purpose. Insertion of 8 x 12-in. (200 x 300 mm) shims of this kind brings about a small angular change. However, undesirable stress concentrations should be guarded against. Solid steel plates are, for this reason, less suitable. If a larger joint is required, the segments should be held in place until an adequate cast-in-place joint is constructed.

# CHAPTER 5 — JOINTS

## 5.1 General

Joints separate the structure into a number of segments. It is generally preferable to use as few segments as possible consistent with transportation and erection economics. The joint surface is usually oriented approximately perpendicular to the centroidal axis of the structure.

The capacity of a joint is a function of the prestressing force normal to the joint and development of friction on the joint face. This Chapter discusses the joint types. A further discussion of design considerations for joints is presented in Chapter 7.

Joints are either wide or match-cast. Wide joints may be cast in place with concrete, dry packed with mortar, or grouted. Match-cast joints are normally bonded with epoxy but dry joints have been used in special instances. For all types of joints the surfaces must be clean, free from grease, oil or other contaminates. Where epoxy bonding is to be used the joint surfaces should be lightly sandblasted while maintaining the match-cast fit.

Post-tensioning tendons crossing joints should be approximately perpendicular to the joint surface in the direction of the smaller dimension of the segment element concerned, i.e., flange or web thickness; but may be inclined in the direction of the larger dimension, i.e., flange width or web depth. This is to minimize any unbalanced shearing force which could lead to dislocation of edge zones at joints.

Holes for the passage of tendons must be located very precisely. Particular care is required to completely seal tendon ducts at joints in order to prevent penetration of joint filling material into the duct, thereby blocking passage of tendons, and also to prevent unsightly leakage at joints during grouting.

## 5.2 Cast-in-Place Joints

In the case of cast in place or other types of wide joints, prior to construction of the joint, the adjacent concrete surfaces should be roughened and kept thoroughly wet, or a bonding agent approved by the Engineer may be applied.

The design width of a joint must allow access for coupling of conduits, welding or lapping of reinforcement, and thorough compaction of concrete. Typical joint widths are equal to the deck slab thickness or one-half of the web width, but not less than 4 in. (100 mm).

The compressive strength of the joint concrete at a specified age should be equal to the strength of the concrete in the adjacent precast segments. High early strength portland cement may be used. Aggregate size should be selected to ensure maximum compaction. Serration of the joint is recommended.

The height of each concrete placement or lift must be limited so that the concrete can be properly consolidated. Ports are normally provided in the joint form for inspection. Formwork should be sealed to prevent leakage of paste. Cast-in-place joints should be adequately cured.

The sharpness of line of assembled segments with wide joints depends mainly on the accuracy of the manufacturing of the joints during erection and less on the accuracy of the segments. Curvature and twisting of the structure may occur within the joints from the post-tensioning unless proper care is taken to prevent it.

## 5.3 Dry-Packed Joints

The width of dry-packed joints should not exceed approximately 2½ in. (65 mm). Mortar should be introduced into the joint in batches not exceeding 10 lbs (4.5 kg) by weight. Each batch should be thoroughly tamped and packed before the next batch is placed. Mortar should be rammed into place using a heavy hammer and ram, or a pneumatic tamper. Containment formwork may be necessary, particularly at the bottom of the joint.

The strength of the mortar in the joint may be lower than that of the concrete in the adjacent segments and this factor may be a critical design consideration for the structure. The use of special high strength mortars may have to be considered. The mortar should have a compressive strength of at least 4000 psi (281 $kg/cm^2$) measured on cubes in accordance with ASTM C-109.[29] If an expansive additive is used the U.S. Army Corp. of Engineers Specification CRD-588[30] may be used. Mortar should be thoroughly mixed and have zero slump. Maximum aggregate size normally does not exceed $\frac{3}{16}$ in. (5 mm).

## 5.4 Grouted Joints

The width of grouted joints should not exceed approximately 1 in. (25 mm). Grouted joints may be made with either gravity or pressure methods. In either case provisions should be made to contain the grout.

When gravity grouting is used, rodding or tamping should be used to consolidate the grout. Problems may be encountered with gravity methods, particularly on horizontal joints. For this reason the grout should consist of approximately three parts of cement, one part of clean sand passing a #16 mesh screen, and a water-reducing agent which minimizes sedimentation. The total water content should be kept as low as practical. Shrinkage compensating additives should be considered.

Pressure grouting of joints requires adequate preparations and equipment that is in good and clean condition. The joint should be tightly sealed in order to sustain the pressure. Pressure grouting is particularly effective when joint forms can be easily used. Joint forms should be vented. At the conclusion of the grouting operation, the vent should be closed, and the pressure increased a minimum of 15 psi (1 kg/cm$^2$) at the vent. Twenty-four hours after grouting, the vent should be opened and filled if sedimentation has occurred.

In both grouting methods venting is of extreme importance. Horizontal joints should be vented at the highest points and at several points on each side. The vents should have provisions for closure which can withstand grout pressure. The strength of the grout in the joint may be lower than that of the concrete in adjacent segments and this factor may be a critical design consideration for the structure. The compressive strength of the grout at a specified age should be at least 4000 psi (281 kg/cm$^2$) measured on cubes in accordance with ASTM C-109.[29] If an expansive additive is used the U.S. Army Corp. of Engineers Specification CRD-588[30] may be used. Non-metallic grout mixes should be used when the grout will be exposed to weather.

## 5.5 Match-Cast Joints

### 5.5.1 Epoxy Bonded Joints

Epoxy resins used for bonding of match-cast joints are high quality products having compressive, tensile, bond, and shear strengths equal or superior to concrete. These products are subject to strict specifications and laboratory certification of their chemical, physical and mechanical properties. Manufacturers' recommendations for successful application in the field should be strictly adhered to.

The three major purposes of epoxies used in match-cast joints are:

**1.** *Epoxy serves as an erection aid.*

**A.** The epoxy, while in its gel form, serves as a lubricant between segments during the fitting process.

**B.** The epoxy serves as a medium for filling up unwanted voids and surface flaws on the jointing surfaces.

**2.** *Epoxy improves the durability of match-cast joints.*

**A.** Complete filling of the joint provides weather tightness.

**3.** *Epoxy improves the structural performance of match-cast joints.*

Since the tensile strength of epoxy itself and the bond strength between epoxy and concrete can be equal or superior to the tensile strength of concrete, the use of a suitable epoxy restores "the tensile strength of concrete" at the match-cast joints. The significance of this is:

**A.** The use of epoxy increases the overload at which the joint opens and makes the magnitude of this equal to the load required for cracking the sections surrounding the joints. Crack development under overloading tends to be similar to that in a monolithic structure. This also improves the tightness of the closure of such cracks upon removal of the load.

**B.** The joints are subjected to secondary effects which make complete restoration of tensile strength desirable. For example, temperature gradients in top slabs of box girder bridges can cause considerable tensile stresses. Also, the distribution of bending moments in the top slab caused by wheel loads is more effective across moment-resisting joints. It is evident that joints without epoxy do not have moment capacity unless there is substantial compression normal to the joint face. Such compression is usually not provided in negative moment areas of box girders designed for zero tension under full loading. For a further discussion of joints see Section 7.4.

Bridge model tests and other structural tests having epoxied joints with large keys have been tested up to failure and were found to perform excellently. Breen stated that "Epoxied joints did not induce any discernable weakness of this type of construction."[31]

If this high-performance epoxy is not provided, the joint or key will be a plane of weakness in the structure. The durability, capacity to sustain overloads, and moment capacity of the individual slabs at the joints are affected, and must be attended to by using alternative methods.

#### 5.5.1.1 Selection of Epoxy

The compressive strength of the epoxy should equal that of the concrete in adjacent segments under any environmental condition that may be encountered during the life of the structure.

Only highly cross-linked epoxy adhesives that have been thoroughly tested in accordance with current ASTM[32] and applicable industry standards should be used. Information which should be obtained from the manufacturer, or be determined by test, must include as a minimum:

(a) Workability, pot life and "open time" at various temperatures.

(b) Compressive strength and modulus of elasticity at various temperatures after curing for at least 7 days at 68 F (20 C).

(c) Direct shear strength of bonded concrete prisms at various temperatures, with notation of whether failure occurs in the joint or in the concrete.

(d) Ability to bond in such a way

that the tensile strength of the concrete is fully restored.

(e) Creep behavior of the epoxy compound.

#### 5.5.1.2 Application of Epoxy

The adhesive should be supplied in preweighed packs or cans of resin and hardener component. Resin and hardener should have different colors.

Components of the epoxy mix should be proportioned and mixed thoroughly until a uniform color is obtained, following the instructions of the manufacturer. Each batch should be assigned a number. The adhesive should be applied immediately after mixing, and the joint surfaces brought together before the pot life expires. A record of the location of the joint where the batch of epoxy is used, weather, temperature, and observed pot life should be maintained. Tests should be conducted to verify uniformity of mixing in the field.

The concrete surfaces which are to be bonded should not be wet; a damp but not dark, shiny surface is permissible. A wet concrete surface may be dried with hot air prior to applying the adhesive.

The adhesive should be applied in a uniform thickness to both surfaces. Some post-tensioning should be applied within the "open time" of the adhesive. If the correct amount of adhesive has been used, a little will extrude from the joint when pressure is applied. It is recommended that a minimum of 30 psi (2 kg/cm$^2$) be applied to the joint. After the stressing has been completed, a wiper should be pushed into each open tendon duct or sheathing to remove or even out any epoxy that may have entered the duct, and to seal any pockets that form at the joint.

In the case of unforseen interruptions, the "open time" may have expired before the segments are fully joined. In such an event the epoxy should be completely removed according to the instructions of the manufacturer. Sandblasting may be necessary. Particular attention to the manufacturer's recommendations and AASHTO[33] is required in cold weather.

### 5.5.2 Dry Joints

The requirements for the use of dry joints are similar to those for epoxy bonded joints but with an even greater emphasis on the quality of the matching faces. Surfaces should be carefully checked to ensure that any high points or other projections are eliminated.

Care should be taken during erection of segments to ensure that joints are completely closed before applying the final prestress.

In bridge construction special treatment is required at the deck level to seal the joints against penetration of moisture. In cold climates where de-icing salts are used special care should be taken to prevent salt penetration through the joints into the tendon ducts. For this reason the use of dry joints is not recommended where corrosive environments exist or where bridge decks are salted.

## 5.6 Metal Joints

Mating pairs of steel plates which are joined by welding, or similar suitable steel connections have been used. However, they should be considered more as connections than as typical joints between precast elements in segmental structures. Special attention is required for welded joints to prevent cracking or spalling of the concrete. Protection against corrosion should be applied to prevent staining of adjacent surfaces and to protect the integrity of welds and contact surfaces.

# CHAPTER 6 — POST-TENSIONING TENDONS

## 6.1 Sheathing

Sheathing is used to form the holes or enclose the space in which the prestressing steel is located. It is usually located inside the concrete section; however, in some cases it has been located outside the section.

The prestressing steel can be placed with the sheathing, or installed after the concrete is placed. The cross section of the sheathing should be adequate to allow proper installation of the prestressing steel and to provide enough passage area for filling the sheathing with grout subsequent to stressing. The ratio of the net area of the prestressing steel to sheathing cross-sectional area should comply with the Guide Specification for Post-Tensioning Materials in Reference 34. For long tendons [more than 100 ft (30 m)] inserted into the sheathing after the concrete is placed, it is recommended that slightly oversized sheathing be used to facilitate installation of tendons. (Note that the sheathing size given in Reference 34 is considered a minimum.)

Sheathing should have sufficient grouting inlets, vent pipes for escape of air, shut-off valves, and drains to allow proper grouting and to avoid accumulation of water during construction in freezing climates. For continuous structures, ducts exceeding 400 ft (122 m) in length should be vented over each intermediate support, and at such locations as shown on the plans. Intermediate vents may not be required for ducts less than 400 ft (122 m) in length.[35]

Sheathing placed inside the concrete should be resistant to damage caused during installation and placing of the concrete and the installation of pull-through tendons. If the sheathing wall has inadequate strength to resist such damages, it may be protected by inserting inflatable duct tubes, steel pipe, etc. during the placement of concrete. The sheathing should be adequate to prevent leakage of concrete into the void, and should be secured against buoyancy.

When semi-rigid sheaths are used, they should bend sufficiently to follow the required tendon profile. All sheaths should be tied either to the reinforcing bar cage or to special supports. Semi-rigid sheathing of 2¾ in. (70 mm) diameter or larger should be tied at a maximum of 5-ft (1.5 m) intervals. Smaller diameter or flexible sheathing should be tied more frequently to minimize wobble.

In certain applications, the sheathing is arranged outside the concrete section either in voids of the section or along the outside of the concrete cross section. In addition to the requirements listed above, all sheathing should be protected against corrosion or made of non-corrosive material such as polyethylene.

Sharp curvature and/or changes of angle should be avoided and sheathing splices should be tightly applied and fastened to facilitate the threading of tendons through the sheathing.

Sheathing may be connected at the joints by:

(a) In the case of match-cast segments, joining the two matching surfaces with the epoxy filler will establish the connection and continuity of the sheathing void.

(b) Telescopic sleeves pushed over the protruding ducts.

(c) Screw-on type sleeves.

(d) Rubber or plastic sleeves.

(e) Gaskets.

In all cases, leak tightness should be assured to avoid the entrance of jointing materials into the sheathing, causing possible blockage. Sheathing protruding from the surface is easily damaged, and must be repaired prior to making the connection. Post-tensioning ducts should be swabbed or wiped to ensure the removal or smoothing out any epoxy that might have leaked into the duct.

## 6.2 Tendon Couplers

Couplers should be designed to develop the ultimate strength of the tendons they connect. Adjacent to the coupler, the tendon should be straight for a minimum length of 12 times the diameter of the coupler unless the curvature is very minor or the strength of the system is determined by tests. Adequate provisions should be made to assure that couplers can move during prestressing. Large coupler void areas should be deducted from gross section areas when computing stresses at the time of prestressing. Enclosures around the couplers must be provided with a vent pipe.

## 6.3 Anchor Plates

The anchor plate is the component of a post-tensioning system which transmits the prestressing force from the tendon anchoring device directly to the concrete. Its function is to distribute the concentrated forces from the anchoring device over a larger bearing area to the concrete. The bearing surface should be perpendicular to the prestressing steel to prevent undesirable secondary stresses of the prestressing steel and to optimize the effectiveness of the anchors. The anchor plate should be of such shape and dimensions as to limit the bearing stresses to those specified in the *Post-Tensioning Manual*,[34] the *AASHTO Bridge Specifications*,[33] or other applicable codes. Mild steel reinforcement behind the anchor plates should be detailed to resist bursting stresses.

### 6.3.1 Anchor Plates Cast into Precast Segments

This is the most commonly used procedure. The concrete should be thoroughly vibrated behind the bearing plate to avoid honeycombing and to achieve proper strength. However, over-vibration may cause segregation of concrete.

### 6.3.2 Anchor Plates Placed Against Precast Surface

When the bearing plate is placed against the hardened surface of the precast element, dry-packed mortar or another suitable material should be used between the plate and the concrete to achieve a good degree of stress distribution under the bearing plate. The strength of the mortar or other material should not be less than the concrete strength on which it bears, and the thickness of the joint should be limited to a maximum of 2 in. (50 mm).

The reinforcement of the concrete underneath the dry-packed mortar or other material should be the same as if the plate was cast into the precast segment.

The preparation of the surface under the bearing plate will depend on the type of plate and placing procedure used.

When the bearing plate is not cast into the concrete, the surface should be clean and free from foreign matter. The surface should be wire brushed to remove all latents and loose concrete chips. If the bearing plate is to be grouted, proper bond should be assured.

## 6.4 Tendon Layout

Attention should be given to the tendon layout to make it compatible with:

(a) Sequence of the segmental construction.

(b) Progressive and subsequent load conditions which the segmental structure will undergo while being erected.

(c) Concrete placement.

(d) Effective section used in design.

It is important to anticipate the secondary effects (stresses due to restrained deformations) which post-tensioning may have on the structure, and how these effects can be influenced by changes in the tendon profile. The effects of concrete creep and changes of the statical system during construction should be considered.

Anchorages, couplers and splices of post-tensioning tendons should preferably not be located in areas where yielding may occur under ultimate load conditions.

## 6.5 Placement and Stressing of Tendons

When tendons are installed in the segments before casting, they are usually coupled together at each joint. This construction method permits stressing of part of a tendon, after installing one or more segments, before the full length is completely installed.

Tendons may also be installed after casting and erection in parital lengths and coupled together at special openings. Usually, however, these tendons are installed full length. It may be noted, that this coupling procedure permits intermediate stressing of portions of the structure, by using tendons of variable lengths, stressing the short ones first and long ones later. The prestressing steel should be free of detrimental rust when installed into the structure. Special attention should be given to the temporary corrosion protection of the prestressing steel until grouted.

## 6.6 Grouting

The purpose of grouting is to provide corrosion protection to the prestressing steel, and to develop bond between the prestressing steel and the surrounding concrete. To accomplish this, the grout should fill all the voids in and around the post-tensioning tendon for its entire length. Grouting materials, preparation of grout, and grouting procedures should follow the Recommended Practice for Grouting of Post-Tensioned Prestressed Concrete[36] wherever applicable.

Grouting of tendons should be completed as soon as practical after tensioning of the tendons. It is recommended that all tendons which are in the same group be grouted within a short period of time. Grout crossing over from one duct into another, which sometimes occurs at a joint, will be easier to detect and remedy if all tendons in the group are grouted in a short interval of time.

As a consequence of this practice, some time may elapse between stressing and grouting of the first tendon in such a group. If the period of time that a tendon is left ungrouted exceeds 20 days,[37] a temporary corrosion protection system for the tendon should be employed.

Grouting is preferably done from the lowest points of the tendon, toward the higher points, where vent pipes should be installed. The grouting should be done slowly and at low pressure to insure full displacement of the air. Highly pressurized and rapid grouting may create entrapped air. The tendon may be flushed with water before grouting to avoid blockage when injecting the grout. Sufficient purging of grout should be allowed to demonstrate that the exiting grout has the same consistency as the grout pumped into the sheathing.

Grouting should not be done if the temperature in the duct is less than 35 F (2 C) or if the surrounding concrete temperature is less than 32 F (0 C).

When grouting vertical tendons the potential of segregation of the grout is greater than with horizontal tendons. To minimize sedimentation and to avoid dehydration and consequent blockage of grout and possible joint leaks, the grout should have the property of inhibiting water separation when under pressure. Additives for this purpose may be used with approval of the Engineer.

Precautions should be taken to reduce the danger of blockage of grout due to the couplers. Couplers are generally housed in a special, enlarged encasement, which should:

(a) Have the same open cross sectional area for passage of grout as the rest of the tendon.

(b) Be provided with a grout inlet and/or vent pipe.

## 6.7 External Tendons

External tendons have been used in segmental construction. Attention should be paid to the strength of the anchorage in relation to the tendon, corrosion protection, and the ultimate moment and shear capacity of the structure.

If external tendons are used, construction details should prevent access of water into the ducts or anchorages.

* * *

# CHAPTER 7 — DESIGN CONSIDERATIONS

## 7.1 General

The analysis and design of segmental structures are essentially the same as for monolithic prestressed concrete structures except that due consideration should be given to the following items:

(a) Redistribution of forces due to creep of concrete, which changes the stresses of the as built conditions, and erection conditions.

(b) Weather-proof tightness and strength of connections and joints.

(c) Storage and handling of segments before and during construction.

(d) Camber, during erection and in the casting bed.

## 7.2 Force Redistribution

In most cases, the statical system of the structure during construction stages is not the same as in the final stage under service load. Redistribution of forces due to creep and shrinkage of concrete should be considered.

As the structural system changes during construction, creep of concrete tends to redistribute the moments and forces in the structure to approach the conditions as if the structure were built totally on falsework. However, depending on environmental factors, concrete properties and construction schedules, the degree of this redistribution varies.

Calculation of the force redistribution may follow the recommendations of ACI Committee 209[38] or other recognized methods.

## 7.3 Handling and Storage of Segments

Handling and storage of precast segments requires special attention so that no warping or bending of individual segments will take place. It is suggested that the Engineer specify the method of support during storage and handling.

## 7.4 Joint Design

The joints between segments should be capable of transferring the compressive, shearing, and torsional forces determined by the design. There should be no tensile stresses in extreme fibers when all possible design loads such as live load, temperature, wind, construction loads, etc. are properly considered. Longitudinal bending moments in the top slab due to wheel loads should be included in the analysis. Temperature variation through the cross section may cause significant stresses in bridge superstructures and should not be neglected.

The joints are subjected to shear stresses. In box girder bridges, the joints are generally perpendicular to the neutral axis. Shear forces act in the plane of the joint, and perpendicular to that plane are prestressing forces. In many precast segmental bridges, the ratio of the prestressing forces and shear forces is seldom less than 5. For a dry match-cast concrete-to-concrete joint, if the friction coefficient is assumed to be 1.0, the resulting safety against sliding would equal 5. Therefore, under those conditions, dry joints of this type would safely transfer the shear even without keys or serrations. If the match-cast joint is filled with epoxy, the "friction concept" applies as well.

Shear in match-cast joints can be safely transferred without keys, using either dry or epoxied joints. The epoxy joint, however, is mostly used because of its other desirable properties. Note that all requirements are satisfied only by using epoxies with good bond capabilities and tensile strength. Tests performed on epoxy joints have demonstrated their adequacy.

### 7.4.1 Shear

Shear-friction provisions are given in AASHTO Specifications[39] and in the ACI Building Code (318-77).[40] Diagonal or vertical web tendons may be used to reduce shear forces or to increase shear capacity; however, attention should be given to the location of these tendon anchorages. If tendons are considered effective for increasing the shear capacity they should meet the applicable provisions of the Specifications[39] or Codes[40] for developing shear reinforcement. If wide joints are used, they should contain non-prestressed shear reinforcement equivalent to that in the adjacent segments.

### 7.4.2 Keys

Keys in the webs and slabs of segments are useful to facilitate alignment and to transfer shear during erection. Two systems are used:

1. A large, single, reinforced key usually designed for transfer of shear caused by a number of segments.

2. A large number of unreinforced small multiple keys which, when properly arranged and designed, are capable of transferring shear forces higher than possible with a single, large key.

The large key is easily designed as a corbel, and leaves ample opportunity for penetration of the matching surfaces by anchors, tendons, etc. The large keys are located in or near the neutral axis of the section. This ensures placement in a compression zone even in case of moment reversals.

The shear capacity of multiple keys depends largely on the two-dimensional stress condition in the keys caused by compression and shear very similar to the dry concrete-to-concrete joint. As in any other joint, the multiple key depends, for its strength, on compression normal to the joint. As a consequence of this, multiple keys located in a precompressed tension zone may have little to contribute to shear transfer under full load, because of a low ratio of normal force to shear force.

### 7.4.3 Compression

As indicated in Chapter 5, the compressive strength of materials used in a joint may be lower than that of the concrete in adjacent segments and this factor may be a critical design consideration for the structure. If a dry joint is used, consideration should be given to the prevention of spalling around the edges of the contact area. It is recommended that dry joints be used only when previous performance indicates acceptable transfer of forces and long-term weatherproofing.

## 7.5 Auxiliary Reinforcement

In addition to the primary reinforcement for transverse bending, shear and torsion, auxiliary reinforcement is normally required in segments for:

(a) Concentrated forces at reactions and at tendon anchorages.

(b) Volume change forces.

(c) Temporary forces imposed during fabrication, transportation, or erection.

(d) Temperature gradient stresses between the top and bottom of the segment and between inside and outside surfaces of exterior webs.

Horizontal and vertical reinforcement should be placed between segments in wide joints filled with mortar or concrete when the transverse thickness of the joint is more than 2 in. (50 mm).

## 7.6 Design of Segments for Box Girder Bridges

For constant depth segmental bridges, span-to-depth ratios from 18 to 25 are currently considered practical and economical. Midspan depth of variable depth girders may have span-to-depth ratios of 40 to 50; span-to-depth ratios of equal to that of constant depth girders are used at the piers.

Simple single cell box girders may be used if the width of the deck is less than 20 percent of the span. This is based on the use of simple beam theory for which the distribution of longitudinal stresses is limited to a deck width per web of 10 percent of the span length. For wider decks, multiple cells are desirable and the webs should be located to reduce moments in the top slab and to minimize transverse bending of the webs. Generally, these moments are computed with sufficient accuracy by considering the box section as a closed rigid frame. Transverse prestressing in the deck slab has often been used.

Deck slab design should also include longitudinal local bending moment in the deck slab due to wheel loads which can cause significant flexural stresses. If curved tendons are used, radial forces due to curvature of tendons should be considered. In some cases tendons are given curvature in addition to the principal tendon curvature at certain locations in order to accommodate better distribution of voids in the concrete or arrangement of anchors; forces due to these secondary trajectories should be carefully evaluated. Fillets or reinforcement may be added for this purpose.

Diaphragms are normally required at abutments, piers and hinges. Intermediate diaphragms contribute very little to load distribution and are not recommended.

Special consideration should be given to the effects of severe environmental exposure.

For more information on segmental box girder bridges refer to the *Precast Segmental Box Girder Bridge Manual*.[37]

## 7.7 Design of Segments for Decked I-Beams

The spanning capability of typical I-shaped sections is extended when the beam and deck are precast integrally.

Moment redistribution due to creep of the concrete should be considered in order to estimate the interaction between the beams and the deck slabs if the structural system changes during construction.

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**NOTE:** Discussion of this report is invited. Please submit your discussion to PCI Headquarters by September 1, 1982.