3.1 Introduction

Construction of segmental concrete bridges involves assembling smaller pieces of concrete members called segments using posttensioning tendons to form a bridge structural system, either superstructure or substructure. These segments can be produced by cast-in-place or precast/prefabricated methods, while the posttensioning system can be bonded, unbonded tendons, or a combination of both. Bonded...
tendons typically consist of cementitious grouted internal tendons, while unbonded tendons could be cementitious grouted or greased, waxed, and sheathed, in the form of external or internal tendons. In segmental bridge design, it is critical to determine the construction means and methods, prior to proceeding with the design. The construction method will greatly affect the outcome of design and tendon layouts, unlike any other type of structures. In most cases, construction loads will also impact the design, material quantity, and details.

The following are some of the important events/milestones in the development of segmental concrete bridge construction from its infancy after World War II to the current state-of-the-art standard of practice.

In 1939, Eugene Freyssinet of France developed a conical wedge posttensioning anchorage system for wires which led to the wide application of posttensioned structures possible. Without posttensioning systems, the segmental bridge construction could not have been realized.

From 1941 to 1949, Freyssinet was the first to apply precast prestressed segmental construction for several bridges at Luzancy over the Marne River east of Paris, France.

The development of a modern long-span cast-in-place segmental bridge, the Lahn Bridge in Baldiustein, Germany, was pioneered by the German engineer Dr. Ulrich Finsterwalder of Dyckerhoff & Widmann AG in 1951. The bridge was constructed with the balanced cantilever method and the segments were cast on a form-traveler attached to the previously cast and stressed segments. Posttensioning was applied to the newly cast segment after the concrete had hardened against the previously stressed segments.

In 1954, French engineer Jean Muller applied for the first time the dry-joint match casting innovation to the construction of a small single-span bridge called Sheldon Bridge in upstate New York, USA, by assembling three precast girder segments and posttensioned them together on site to form a single span girder in order to facilitate the transportation of the girders and improve the speed of construction. It is much easier to handle and transport small pieces of girders than a long girder.

In 1962, Muller for the first time applied precast segmental box girder using epoxy-coated match cast joints between segments and posttensioned them together for the construction of the Choisy-le-Roi Bridge over the Seine River in Paris, France. The bridge was constructed by Campenon Bernard, a general contractor where Muller was the technical director of the company.

Since then, precast segmental construction has gained popularity over cast-in-place due to its speed of construction, mass production of segments in the casting yard, better quality control, ease of transportation, and overall economical structure. However, precast segmental technology is not suitable for every bridge project; it depends on the size of the project, span length, location, local standard practice, and so on. Under certain circumstances, cast-in-place segmental bridge construction is still popular, such as for a very long span bridge over river crossing or deep valley (see Section 3.4 on Conceptual Design for more discussions on selecting bridge types).

This chapter presents a practical knowledge to practicing engineers, owners, and graduate students on a complete spectrum of segmental concrete bridge design from conception to final design, including construction means and methods.

In Sections 3.4 to 3.8, the AASHTO LRFD Bridge Design Specifications (AASHTO 2012) on segmental bridge design provisions are extensively referenced (AASHTO LRFD Sections 4 and 5). AASHTO LRFD adopted segmental bridge design provisions from AASHTO Guide Specifications for Design and Construction of Segmental Bridges, 2nd Edition, 1999. The guide specifications have served as design guidelines and specifications and construction on segmental concrete bridges for many years since the first edition in 1989, prior to AASHTO LRFD Design Specifications.

Aside from design and construction, durability of segmental concrete bridges was discussed in Section 3.9 in order to fulfill the intended service life of the structures, minimize maintenance cost, and to build awareness on the importance of workmanship and quality control during the whole process of building a segmental bridge from design to the completion. Good detailing and high standard industry practice is an important part of a design process in order to maintain constructability during construction, structural integrity, and durability during its service life.
3.2 Structural Material

3.2.1 General

The basic structural materials for posttensioned segmental concrete bridges are very similar to posttensioned structures in general, which consist of high-strength concrete, high-strength prestressing steels, and ordinary reinforcing steel, including grout in the duct. Earlier, prestressed concrete structures failed due to lack of understanding of creep and shrinkage and long-term loss of prestress. Freyssinet was credited with the use of the first successful prestressed concrete by the application of high-strength steel to counteract the creep and shrinkage of concrete. Ordinary reinforcing steel also plays an important role in supplementing concrete and high-strength prestressing steel as the primary structural materials. Without ordinary reinforcing bars, a posttensioned prestressed concrete structure will not be able to function properly. For instance, local zones around the anchorages may crack, without ordinary reinforcement confinement in the form of spiral; and ordinary reinforcement will also contribute to the shear-carrying capacity in the webs, in addition to concrete and posttensioned inclined tendons. Ordinary reinforcement also plays an important role in partially prestressed concrete structures in controlling flexural crack width in concrete and contributes in flexural ultimate capacity of the structure.

There are two types of tendons used in segmental bridges, namely bonded and unbonded tendons. The earlier segmental bridges have mostly used grouted corrugated metal duct (internal), but since 2003 most owners in the US have switched to corrugated plastic duct made of either polyethylene (PE) or polypropylene (PP) for better tendon corrosion protection. For external tendons, smooth PE ducts are generally specified in the United States. European countries have also adopted petroleum grease, wax, and monostrand greased and sheathed tendons as another alternative of corrosion protection system.

3.2.2 Concrete

High-strength concrete for posttensioned bridges is required in posttensioned structures because of high compressive stresses transferred around the anchorages to the concrete members and precompression of the posttensioning forces to the member section. Without high-strength concrete, prestressed concrete is not efficient and economical. Typically, a minimum of 5,000 psi (35 MPa) concrete strength at 28 days is required in prestressed concrete structures. In addition, higher-strength concrete results in higher modulus of elasticity. This is a preferred concrete property to minimize long-term creep deformations. With the advancement of concrete technology, producing 10,000 psi (69 MPa) to 12,000 psi (83 MPa) concrete is a common practice nowadays in the precast industry. Therefore, more and more precast prestressed concrete structures are being constructed with concrete strength equal to or higher than 8,000 psi (55 MPa). For an aggressive environment, it may be necessary to select a mix design with high strength in combination with high-performance concrete (low permeability). High-performance concrete will improve the durability and protect the posttensioned tendons from corrosion. In case of concrete members with highly congested reinforcing bars such as diaphragms, blisters and deviators, self-consolidating concrete (SCC) is recommended in order to avoid honey combing.

3.2.3 Posttensioning Systems

PT anchorage systems in posttensioned bridges are a proprietary system. Any one of these systems such as VSL, DSI, Freyssinet, BBR, Schwager Davis, and other systems can be found in the industry/market in the United States. The PT anchorage systems are designed in different shapes, sizes, and material. In general, a basic posttensioning anchorage system comprises a bearing plate, trumpet, wedge plate (anchor head), grout cap, and grout ports as shown in Figure 3.1.
In 2003, the Florida Department of Transportation (FDOT) required an additional vertical grout port/vent located above the trumpet to facilitate postgrouting inspection and permanent grout cap in its posttensioning specifications. Notice the differences between the older and newer generations of PT anchorages systems as shown in Figures 3.2 and 3.3. Many other state DOTs have adopted PT anchorages similar to FDOT requirements. For the new anchorages, the inspection access will be a lot simpler through the vertical grout port.

In general, posttensioned bridges built in the United States consisted of grouted internal tendons, grouted external tendons, or a combination of the two.

Internal tendons are located inside the structural concrete section and are housed in corrugated metal ducts or corrugated plastic ducts and are bonded to the structural concrete by means of cementitious grout as shown in Figure 3.4. The plastic corrugated duct may be made from high density polyethylene (HDPE) or polypropylene (PP). The high-strength steel tendon could be strands, wires, or bars. External tendons are typically located outside the perimeter of the concrete section and are housed in HDPE smooth duct and filled with cementitious grout. External tendons are not bonded with the concrete structural section (see Figures 3.5 and 3.6). In Europe the external tendons are also filled with flexible filler material, such as petroleum grease and wax.

### 3.2.4 Prestressing Steel

There are many forms of high-strength prestressing steel types in the industry worldwide that can be utilized for segmental and posttensioned bridges such as nineteen-wire strands, compact strands, two and three-wire strands, oval deformed bars, single wire, and so on. However, only two types of...
prestressing steel are commonly used in the United States, as shown below. Parallel wires were used in some older bridges in the past.

1. Uncoated seven-wire stress-relieved for prestressed concrete conforming to ASTM A416-90. It is recommended to use low relaxation for segmental bridges.

Material Properties:
- Ultimate tensile strength ($f_{pu}$): 270 KSI (1,860 Mpa)
- Yield strength ($f_{py}$): 243 KSI (1,674 Mpa)
- Apparent modulus of elasticity: 28,500 KSI (197,000 Mpa)
2. Uncoated high-strength steel bar for prestressed concrete according to ASTM A722-90. It is recommended to use Type II (deformed) bar for segmental bridges.

Material Properties:
   a. Ultimate tensile strength \( (f_{pu}) \): 150 KSI (1,035 Mpa)
   b. Yield strength \( (f_{py}) \): 120 KSI (828 Mpa)
   c. Modulus of elasticity: 30,000 KSI (207,000 Mpa)

3.2.5 Cementitious Grout

Grout material consists of a mixture of Portland cement, mineral additives, admixtures, aggregates, and water. In bonded posttensioning tendons, grout serves as the primary corrosion protection of prestressing steel, in addition to forming a good bond between prestressing steel and its surrounding concrete. Type I or Type II cements according to ASTM C150/C150M can be used for grout mixture. For slower release of heat hydration application, Type II cement is used. Mineral additives could be Class C and Class F fly ash, Grade 120 slag cement, or silica fume. The role of admixtures in grout is to improve set control, reduce water, bleed control, air entrainment, corrosion inhabitation, and pumpability. Some grout contains fine sands, but sand is optional. Clean potable water is used for the grout mixture.

Posttensioning Institute (PTI) Grouting Specifications, 2nd Edition, 2012 listed four types of grout:
   - Class A: Nonaggressive exposure such as indoor or nonaggressive outdoor.
   - Class B: Aggressive exposure such as wet/dry cycle, marine environment, and deicing salts.
   - Class C: Prepackaged for either nonaggressive or aggressive environments.
   - Class D: Engineered grout.

The specifications also specified the material properties required for each type of grout, including the acceptance and test criteria and methods. Although most owners have their own grout specifications, PTI specifications are widely referenced and accepted.

3.2.6 Ordinary Reinforcing Steel

Deformed and plain billet-steel bars for concrete reinforcement conforming to ASTM A615.

Material properties:
   - Yield strength: 60 KSI (400 Mpa)
   - Modulus of elasticity: 29,000 KSI (200,000 Mpa)
Some owners have specified solid stainless steel reinforcing bars conforming to ASTM A955/A955M for structural elements located in the extremely corrosive environment.

### 3.3 Construction Methods

#### 3.3.1 Balanced Cantilever Construction

Free cantilevering is a method of construction used to build outward from a fixed point to form a cantilever structure, without temporary support, using staged construction as shown in Figure 3.7. Definition of “cantilever” from Webster’s Dictionary is: “A rigid structural member projecting from a vertical support, especially one in which the projection is great in relation to the depth, so that the upper part is in tension and the lower part in compression.” Another meaning of “cantilever” is “bracket.”

When two opposing free cantilever structures are attached as a single structure and erected at the same step, it is termed “balanced cantilever construction method” as shown in Figure 3.8.

It is believed that the idea of cantilevering in bridge construction originated in the ancient Orient. Shogun’s Bridge located in Nikko, Japan, is the earliest recorded cantilever bridge, which dates to the fourth century. The Wandipore Bridge, shown in Figure 3.9, was constructed in the seventeenth century using timber cantilever members with the drop-in span girder in Bhutan, between India and Tibet (Petroski, Henry. 1995).

![FIGURE 3.7 Cantilevering construction method.](image1)

![FIGURE 3.8 Balanced cantilever construction method.](image2)
In the steel bridge construction industry, steel trusses were successfully erected using the cantilevering method by the end of the nineteenth century with construction of the Firth of Forth Bridge in England (Figure 3.10) and the Quebec Bridge over the Saint Lawrence River.

The application of the cantilevering method to cast-in-place reinforced concrete bridges took place for the first time with the construction of a 223 ft (68 m) span bridge across the Rio de Peixe in Brazil in 1930. However, the cantilevering method for reinforced concrete was not successful due to excessive deflection and heavy reinforcing required. Dr. Ulrich Finsterwalder of the firm Dyckerhoff & Widmann AG (DSI International) successfully applied posttensioning to a cast-in-place concrete bridge using the balanced cantilever method with construction of the Lahn Bridge at Balduinstein in Germany in 1950–1951, after World War II (see Figure 3.11). The bridge is fixed at both ends and has a span length of 203.65 ft (62.0 m). This bridge is considered the pioneer of modern long-span balanced cantilever segmental concrete bridge construction.

After successful completion of the Lahn Bridge, the system was improved over the years and has gained popularity for construction of long-span bridges across the world (see Figure 3.12). Cast-in-place balanced cantilever bridges are especially suitable for construction of long spans over deep valleys and
rivers where placing temporary supports is not possible or cost prohibitive. The only drawback of the system is the time required for superstructure construction. For instance, the time taken to cast every increment on site can be considered slow in comparison to precast concrete. In cast-in-place balanced cantilever construction, a starter segment is first constructed over a pier column. The starter segment over the pier is called a pier-table as shown in Figure 3.13. From this starting point, the bridge can be built from a single pier or multiple piers at the same time using form-travelers moving toward mid-span (see Figure 3.14). At mid-span, the two adjacent cantilever tips are connected to make a continuous structure with a closure pour segment. Typically a form-traveler and steel strong backs are attached to both cantilever tips to prevent differential movement during the closure pour (see Figure 3.15). These attachments can also be used to correct horizontal misalignment as well as elevation of both cantilever tips.

California’s Pine Valley Creek Bridge rises 450 ft (137.2 m) above the valley floor, and is 1700 ft (518.2 m) long including five spans of 270 ft (82.3 m) + 340 ft (103.6 m) + 450 ft (137.2 m) + 380 ft (115.8 m) + 270 ft (82.3 m) and was the first cast-in-place segmental concrete bridge built in the United States in 1974.
FIGURE 3.13  Pier table of a cast-in-place balanced cantilever construction.

FIGURE 3.14  Segment placement supported by form-travelers.

FIGURE 3.15  Strong back across a closure pour.
In classical cantilever bridge construction, segments are placed symmetrically from the pier table with typical segment lengths ranging from 10 to 16 ft (3 to 4.9 m). Segment lengths longer than 16 ft (4.9 m) are not recommended due to longer segments producing large out-of-balance loads during construction. In addition, 16 ft segment form-travelers are widely available in the market, and can often be reused without ordering a new one. In special cases, cantilevering with form-travelers longer than 16 ft has been accomplished.

After successful construction of the first modern cast-in-place balanced cantilever (Lahn Bridge) in Germany in 1950–1951, Choisy-le-Roi Bridge over the Seine River near Paris in France was the first modern precast balanced cantilever bridge constructed using match cast epoxy joint in 1962–1964. The bridge was constructed by the Campenon Bernard contractor and designed by Jean M. Muller. Some advantages of precast segmental bridge over cast-in-place construction are the speed of superstructure erection, less creep and shrinkage effect, and better quality control when casting segment in the casting yard. In fact, bridge construction using posttensioning precast segment pieces was pioneered by Eugene Freyssinet in 1944 with the construction of Luzancy Bridge over the Marne River in France. Precast segmental construction was invented to overcome the slow construction schedule of the cast-in-place construction method. Other advantages of precast segmental construction are mass production in the casting yard, better curing system, independent of weather condition, ease of transporting the segments, flexibility in selecting erection equipments, and overall economical structure.

Precast segmental construction requires a casting yard to accommodate construction materials, casting cells (see Figure 3.16), reinforcing bar jigs, concrete plant, survey towers, curing facilities, segment transporter, segment storage site (see Figure 3.17), offices, and material testing facilities. The precast segments are stored on site at least one month prior to delivery to the project site for placement. Typically, the segments are match cast in 10 ft (3 m) to 12 ft (3.65 m) long segments for ease of handling and transportation. It is important to limit the segment weight to about 60 (534 KN) to 80 t (712 KN), since it impacts the erection equipment capacity to lift and place the segment in place. Therefore, it is common to split the pier segment into two segments in order to reduce the segment weight as shown in Figure 3.18. Depending on the site condition and the size of the project, segment erection can be accomplished with the following equipment:

1. Ground based crane
2. Overhead launching gantry (Figure 3.19)
3. Segment lifter (Figure 3.20)
4. Beam and winch

![FIGURE 3.16 Casting yard.](image-url)
FIGURE 3.17  Segments storage site.

FIGURE 3.18  Split precast pier segment.

FIGURE 3.19  Segment erection with overhead launching gantry.
3.3.2 Cast-in-Place on Falsework

Cast-in-place on false-work construction method is building a bridge superstructure on false-work supported directly on the ground for the entire length of the bridge. It is also common to construct the structure in stages/segments. Therefore, it could be considered as cast-in-place segmental construction. This type of construction is suitable for superstructure with complex geometry, relatively short columns, short to medium span length, and good soil conditions. The temporary false-work is removed after posttensioning is complete. Cast-in-place construction on false-work is relatively slow and labor intensive. The cast-in-place on false-work is depicted in Figure 3.21.

3.3.3 Span-by-Span Construction

Span-by-span construction method is typically meant for erection of precast segments by posttensioning the segments for the whole span. Each segment is temporarily posttensioned with posttensioned bars against the adjacent segment after epoxy glue is applied at the match-cast joint between segments.
Some of the earlier span-by-span bridges have no epoxy applied at the joints. This type of joint is called dry-joint. However, precast segmental with dry joint is no longer allowed in the United States due to water leaking from the deck resulted in durability concern, although some countries are still using dry joint. The permanent tendons typically consist of external tendons entirely (Figure 3.22) or a combination of external and internal tendons. Span-by-span bridge construction can also be designed as a continuous structure up to ten spans, by splicing the longitudinal tendons at the diaphragms. For the external tendons, deviators are required in between the piers (see Figure 3.23). The span-by-span segment erection can be done either using an overhead launching gantry (see Figure 3.24) or underslung gantry. In the case of underslung gantry, support brackets are needed at the columns to support the gantry.

![External tendons inside a box girder.](image1)

**FIGURE 3.22** External tendons inside a box girder.

![Typical deviator.](image2)

**FIGURE 3.23** Typical deviator.
3.3.4 Incrementally Launched

The incremental launching bridge construction method has been used for steel bridge erection for many years. This is not surprising since the steel material can handle both tension and compression well, which is not the case for concrete structures. The first concrete incrementally launched bridge was applied to the construction of a reinforced concrete bridge over the Rio Caroni in Venezuela, South America, in 1962. Soon after that, the first modern prestressed concrete incrementally launched bridge was constructed in 1965 at the Inn Bridge in Kufstein, Austria. Professor Dr. Fritz Leonhardt and his partner Willi Baur were credited with the development of both bridges. Since then, they have designed many posttensioned concrete incrementally launched bridges, and hold a patent for the method in Germany. Many posttensioned concrete incrementally launched bridges have been built around the world, but the construction method has seen few applications in North America.

The basic idea of the incrementally launched bridge is very simple. The bridge is constructed in successive short segments on a stationary casting form located behind one of the abutments, as shown in Figure 3.25. Typically, each segment is about 50 (15) to 80 ft (25 m) in length. The first segment is cast and posttensioned with a steel-launching nose attached to the segment end nearest the abutment. Subsequently, the form is lowered and the first segment is launched by being either pushed or pulled forward. A new segment is cast against the first segment and then posttensioning is stressed to connect the two segments together. The second and first segments are then ready to be launched forward. The construction sequences are repeated until the first segment reaches its final position at the other abutment. The advantage of this construction method is that the launching of the superstructure can be done over the piers from abutment to abutment without disturbing the area below the bridge.

During the conceptual design phase of a project, the advantages and disadvantages are carefully evaluated for each construction alternative to determine the most economical solution for a particular site. Incrementally launched bridges have their own set of interesting features.

Some advantages of incrementally launched bridge are as follows:

1. Requires short casting form of about 100 ft (30 m) long and less heavy equipment.
2. Allows for the ability to cast segments during inclement weather and through the winter by providing shelter and insulation over the casting form.
3. Provides better quality control, similar to precast segment.
4. Possesses simple geometry control.
5. Increases efficiency by repetitive works.
6. Requires no segment storage and transportation.
7. Relatively fast construction (average one segment per week).
8. Superstructure erection can be done over the piers without ground-based lifting equipment.
   Therefore, no maintenance of traffic required and suitable for bridge construction over railways, rivers, valleys, and soft soil conditions.
9. Suitable for top–down construction over environmentally sensitive areas.
10. Requires no temporary false-work.
11. Reinforcing bars are continuous across the segment joints.

Incrementally launched bridges require launching equipment, which consists of a launching nose, hydraulic jacks, pushing or pulling devices, temporary sliding bearings, and lateral guides.

In order to reduce the cantilever bending moment during launching prior to reaching the pier top, a steel launching nose is attached on the forward face of the first segment. The steel launching nose consists of two steel plate girders or two steel trusses, each attached to the leading end of the concrete box girder webs by posttensioned bars embedded in the webs (Figure 3.26). Both of the steel plate girders or trusses are laterally braced together. A light and sufficiently stiff nose is preferred. The nose length is about 60%–80% of the longest span length of the bridge.

Hydraulic jacks at the front of the nose facilitate jack-up of the nose as it reaches the piers or temporary supports (Figure 3.27). The jacks are loaded to a predetermined force to provide support for the launching nose and reduce the bending moments in the girder. When the girder reaches the support, the vertical geometry can be further adjusted to proper grade. A mast and stay cable system can also be used instead of a launching nose to reduce the bending moments in the front segments.

The superstructure can be moved forward, by pulling the superstructure using hydraulic jacks reacting against the abutment where the casting form is located, as shown in Figure 3.28. Another way of launching the superstructure is by pushing the superstructure using a special device, as shown in Figure 3.29.
FIGURE 3.26  Launching nose and segment connection. (Courtesy of C2HC Alliance, Australia.)

FIGURE 3.27  Hydraulic jack at launching nose’s tip. (Courtesy of C2HC Alliance, Australia.)

FIGURE 3.28  Bridge pulling device. (Courtesy of VSL International.)
3.3.5 Full-Span Erection

Full-span erection method is another form of span by span erection for short span bridges up to about 120 ft (36 m). The difference with the span-by-span erection is that the girder is cast in one piece for the whole span, instead of in short segments. The full-span erection is ideal for a very long viaduct such as structure for high speed rail project as shown in Figure 3.30. The girder transportation and girder placement can be handled by one erection equipment. The girder transportation usually can be done over the completed spans. The advantages of full span erection are that it is simpler to produce the girder and the speed of erection. However, the disadvantage of this erection method is dealing with a very heavy girder. It is important to study the optimum span length for the viaduct by considering the availability of the erection equipment.

3.3.6 Precast Spliced U Girder

Precast spliced U girder bridges have been gaining popularity recently for medium-span length bridges, especially horizontally curved bridges (see Figures 3.31 and 3.32). Several long segments of pretensioned or posttensioned tub girders are posttensioned using draped internal tendons in the web to
Segmental Concrete Bridges

form a continuous multi-span girder from end to end. The joints between the girders are cast-in-place concrete. The diaphragms are typically cast at the splice locations. These diaphragms are reinforced concrete or posttensioned transversely. A temporary support is provided at the cast-in-place joint locations to stabilize the structure until the girders are made continuous. In reality, the spliced girder construction concept is also a form of segmental bridge construction using longer segments. The segments are typically lifted with a ground-based crane. This option is particularly attractive for a span range of 150 to 250 ft.

FIGURE 3.31  Precast spliced U girder bridge during erection. (Courtesy of Summit Engineering Group.)

FIGURE 3.32  Precast U girder supported on temporary false-work. (Courtesy of Summit Engineering Group.)
3.4 Conceptual Design

3.4.1 Span Configuration

In the conceptual design stage, span arrangement and configuration should be closely studied first by considering the site location of the bridge. A bridge crossing over a navigable waterway is very much dictated by the horizontal and vertical clearance required. It is also important to know the soil condition and landscapes (e.g., over water, land, valley, or mountainous area). For segmental concrete bridges uniformity of the span lengths is critical in order to maximize the benefit of precasting the segments. The more uniform the span distribution, the more economical the bridge. It is also preferable to have an uneven number of spans from the architectural point of view. Leonhard discussed different approaches (See Figure 3.33) for deep V shape valley and shallow valley conditions (a,b and c,d views respectively).

For deep valley and steep slope and unstable soil conditions, it is not a good practice to place many piers with short span lengths. Therefore, the number of piers should be reduced and the span length increased. Notice also the shape and size of the piers. For shorter span length, the lateral pier dimensions should be slender in order to reduce the wall view effect from an oblique view. For shallow valley, it is important to consider the $L/H$ ratio of the opening between two piers, where $L$ is the span length and $H$ is the pier height. It is preferable to have an $L/H$ ratio equal to or greater than 1.5. The end spans should be less than the typical span length (60% to 80% of typical span length) in order to achieve an efficient design.

![Span distribution](image-url)

Once the typical span length is selected, use the following chart (Figure 3.34) to determine the kind of construction method that is suitable for the particular span. Of course, this is only one of the parameters to consider when selecting the construction method.

### 3.4.2 Span-to-Depth Ratios

For spans up to 250 ft (75 m), constant depth section is typically utilized. However, when the span length is larger than 250 ft, a variable depth section is more economical and efficient. Span length over depth ratio ($L/D$) plays an important role in conceptual design. The preliminary section depth is selected on the basis of the $L/D$ ratio rule of thumb to establish the superstructure structural depth. The initial depth selected is continuously refined in the preliminary and final design. Figure 3.35 shows the preliminary $L/D$ ratio for constant depth section of simply supported girder and continuous girder. Figure 3.36 shows the $L/D$ ratio for a variable-depth girder.

### 3.4.3 Proportional Sections

After all structural components of the bridge have been roughly determined, including shape and exterior dimensions, the conceptual design process continues to study the overall proportional dimensions and shape compatibility and suitability of the proposed structure at a particular site. Typically, several alternatives are studied by varying span configurations, superstructure types, materials, construction means and methods, and the approximate cost for each alternative. In most cases, rendering in three dimensions (3D) (See Figure 3.37 for a 3D rendering) is required so that the owners can also participate in the decision making on the preferred alternative.

### 3.4.4 Structural System

As the conceptual design progresses to a more advanced stage, the next step is to study the structural system of the bridge. The final bridge structural system is studied carefully, particularly the stability during construction and how the statical system changes from stage to stage of the construction until the bridge is completed. This is the nature of segmental bridges, unlike other types of structures such as reinforced concrete or steel conventional types of bridges. The erection and stitching of the segment pieces require many stages and the structural system is constantly changing from time to time until completion.
Span/Depth ratio: \( \frac{L}{D_S} = 15 \text{ to } 18 \)
\( \frac{L}{D_M} = 35 \text{ to } 45 \)

**FIGURE 3.35**  Span over depth ratio for constant-depth bridge girder: (a) simply supported girder; (b) continuous girder.

Span/Depth ratio: \( \frac{L}{D_S} = 15 \text{ to } 18 \)
\( \frac{L}{D_M} = 35 \text{ to } 45 \)

**FIGURE 3.36**  Span over depth ratio for variable-depth bridge girder (for span \( \geq 250' \)).

**FIGURE 3.37**  Rendering of Cincinnati Airport Ramp. (Courtesy of Parsons Brinckerhoff, Inc.)
Figure 3.38a–l shows some of the structural system possible for segmental bridges from simply supported girder to the most complicated portal frame system. Although providing in span hinges is theoretically possible, (Figure 3.38e–h) practically such in-span hinges should be avoided for segmental bridges. The earlier cast-in-place balanced cantilever bridges in Europe had adopted midspan hinges in their bridges. It was later discovered that those bridges had suffered maintenance problems due to excessive deflection at the midspan hinges due to long-term creep and shrinkage (Guyon 1972). The subsequent generation of segmental bridges has eliminated midspan or other in-span hinges. If for some reason, the midspan hinges cannot be eliminated, it is necessary to provide a strong back over the hinge to avoid excessive long-term deflection due to creep and shrinkage.

**FIGURE 3.38** Possible structural systems.
3.4.5 Aesthetic Aspect

Segmental bridge construction is not only popular for long span bridges over rivers and canyons but also in urban and city interchanges and for transit structures such as light rail and high-speed rail guideway structures. Aesthetic value has become one of the most important parameters in selecting the bridge structure. The owners and stakeholders are placing more and more importance on aesthetically pleasing structures. Fortunately for segmental box girder bridges, the shape of the box girder is already inherently attractive. The architect should properly design the pier shape to ensure that the overall look of the bridge is compatible and suitable for a particular location. Several samples of this are shown in Figures 3.39 through 3.42. The structural engineers view aesthetics from a different angle. Structural engineers consider aesthetics from the functionality of each bridge element from superstructure to substructure and the logical flow of forces subjected to the bridges, from live loads, dead loads, seismic load, wind loads, temperature loads, long-term creep, and shrinkage of concrete and ship impact, including constructability and durability. A successful bridge design involves the close collaboration of planners, structural engineers, architects, contractors, and the stakeholders/owners.
3.5 Deck Design

The top deck of a box girder is subjected to complex external forces, static and dynamic loads, thermal gradients, and creep and shrinkage effects. Proper consideration should be given to these effects to prevent cracking and deterioration. De-icing chemicals and freeze–thaw action should also be considered in design to counteract degradation.
Deck replacement is not only costly but also results in inconvenience to the traveling public. For segmental bridge superstructures, deck replacement is not practical and almost impossible to do without closing the entire bridge. Therefore, when designing decks for segmental bridges, it is always a good strategy to be conservative/robust and allow for reserved capacity.

Studies have shown that transverse posttensioning of top decks improves long-term deck durability and results in low life cycle cost (Posten, Carrasquillo and Breen 1987). It is recommended that for all posttensioned box girders the top deck be transversely posttensioned, even for short overhangs. For bridges not subjected to freeze–thaw action and de-icing chemicals, at least the deck should be partially prestressed. The top deck should be designed using elastic methods and then checked for ultimate limit states, not the other way around.

In general, it is standard practice to select a minimum top deck thickness of 8 in (200 mm), although AASHTO-PCI-ASBI Standard Sections Committee recommends a minimum deck thickness of 9 in (230 mm).

### 3.5.1 Design Approach

To correctly represent the final system of the box girder, a three-dimensional analysis incorporating all loads with proper boundary conditions is needed. Owing to the complexity of this type of analysis, in particular, the application of prestressing to three-dimensional systems, this is seldom done. In lieu of this complex analysis, it is common practice to model the box as a two-dimensional (2D) plane frame of unit length, as shown in Figure 3.43. If the thicknesses of the web and bottom slab vary along the length of the bridge, several 2D frames may have to be analyzed in order to obtain a more representative interpretation of these varying cross-sectional properties. The 2D frame model allows for load distribution to the webs and slab members relative to their stiffness.

A typical 2D frame model is assumed to be supported at the lower end of the webs as shown in Figure 3.43. While it could be argued that different boundary conditions exist for this model, this simplified assumption produces reasonable results.
The design loads considered in transverse design include, but are not limited to

- \( DC \) = Dead load of structural components and nonstructural components, such as traffic barrier wall
- \( DW \) = Dead load of wearing surface or future wearing surface and utilities, if any
- \( LL \) = Live load
- \( IM \) = Dynamic load allowance
- \( PT \) = Primary prestressing forces
- \( EL \) = Miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
- \( TG \) = Thermal gradient (± 10°F differential between the inside and outside of box girder) Note: currently not required by AASHTO LRFD Design Specifications, but commonly done in standard practice
- \( PS \) = Secondary forces from posttensioning
- \( CR \) = Creep effect of concrete
- \( SH \) = Shrinkage effect of concrete

Secondary forces of posttensioning are included in ultimate limit state load combinations with a load factor of 1.0.

In addition to service and strength limit state load combinations, the deck design should be checked for construction load combinations, such as segment lifting, construction equipment, and segment stacking (see LRFD Article 5.14.2).

### 3.5.2 Live Load Analysis

When a static concentrated load is applied on a deck, the deck will deflect transversely as well as longitudinally, similar to the structural behavior of a two-way slab. The load distribution becomes more complex when multiple point loads are applied to the deck, such as a truck load. When the structural model is simplified to a 2D frame model, as stated in Section 3.5.1, it is important to obtain the resulting 3D forces to the 2D model.

Commonly, there are two ways of handling live load distributions in the transverse direction:

1. In the past, influence surfaces from Pucher or Homberg Charts have been extensively used in box girder transverse design. These charts are based on elastic theory of plates (homogeneous and isotropic). Some charts are valid for constant depth plate thickness and some for variable depth plate thickness with a parabolic soffit. Depending on the boundary conditions of the selected plate, the dimensionless charts provide bending moments per unit length at the fixed end and midspan only. The fixed end moment (FEM) is then applied as external forces to the 2D frame. The bending moments between supports are approximated by interpolation. The method has limitations for haunched deck slabs, regarding the support depth over midspan depth ratio. This method is approximate and can be useful for preliminary design.
2. A more accurate method is based on a partial 3D finite element model of the box girder. The term “partial” implies that the entire bridge superstructure need not be modeled; rather it should be interpreted as a partial length of the box that will be long enough to include three-dimensional effects. From this model, influence lines can be generated at any section of interest. The influence lines should be generated using a line load consisting of front and rear wheels of a design truck. Since general finite element programs are readily available presently, it is recommended that this method be used for final design. It should be noted that, theoretically, a continuous vehicle barrier could be incorporated into this model to further distribute live load longitudinally. However, owing to discontinuities of the barrier and uncertain future quality, this edge stiffening effect is neglected and not recommended.

It is very important that the live load configuration be strategically placed in order to produce the worst condition (see Figures 3.44 and 3.45). Some common points where stresses are checked include the following:

- Maximum negative bending moment at the root of deck overhang
- Maximum positive and negative bending moments at the center line between two webs
- Maximum negative bending moment in the top deck at the interior face of the webs
- Maximum negative and positive bending moments in the webs and bottom slab
- Maximum negative moment in the deck overhang where the taper begins

Figures 3.46 through 3.51 show typical influence lines corresponding to the above locations using method 2.

In the AASHTO LRFD Specifications (AASHTO 2012), only the effect of a design truck (or tandem) is to be considered for transverse design and no lane loads (see Article 3.6.1.3.3).

### 3.5.3 Posttensioning Tendon Layout

Posttensioning in the transverse direction typically consists of three to four 0.5- (13) or 0.6-in (15 mm) diameter strands per tendon passing through the top slab and anchored at the face of the overhang on each side of the box girder. These tendons are usually housed in flat ducts due to the thin top slab. To efficiently utilize the tendon, it should be suitably profiled for maximum structural efficiency.
A typical tendon is generally anchored at mid height of the slab at wing tips and then gradually rises to a level above the neutral axis of the deck over the webs. This helps the tendon resist the negative moments at the webs. The tendon then gradually drops to a level below the neutral axis of the top slab near the centerline of the box girder in order to resist the positive bending in that region. Figure 3.52 shows one example of the tendon path.
Longitudinally, the tendon spacing is determined using the appropriate service and strength limit state checks. The maximum spacing of tendons is typically restricted to 4 ft (1.2 m) in an effort to limit shear lag effects between anchorages. However, it is a good practice to space transverse tendons based on 30° stress distribution from the anchorage point on each side of the tendon considered. If maximum tendon spacing is not addressed, zones near outside edges of the slab may be without effective prestressing.

3.5.4 Summary of Design Forces

The design forces obtained from the 2D frame analysis and 3D live load influence lines may be combined in a spreadsheet using the LRFD Service Limit State and Strength Limit State combinations. The maximum tensile and compressive stresses at each predetermined section in the top slab are summarized and compared to the stress limits at Service Limit States specified in AASHTO-LRFD Bridge Design Specifications (AASHTO 2012). The prestressing force is usually estimated in preliminary hand calculations, and then analyzed in a 2D time-dependent run using proprietary software. All other loads are incorporated into the 2D model, except live loads. The results are then compiled in a spreadsheet to check stresses. By varying the prestressing force, the combined stresses of service limit states are
Segmental Concrete Bridges

Moment per wheel line (kip-ft/ft)

Distance from centerline box girder (ft)

HL-93 truck values—no impact, unfactored
Tandem values—no impact, unfactored

FIGURE 3.49 Live load influence lines for moment at top of web.

Moment per wheel line (kip-ft/ft)

Distance from centerline box girder (ft)

HL-93 truck values—no impact, unfactored
Tandem values—no impact, unfactored

FIGURE 3.50 Live load influence lines at outside wing/thickness transition.

Moment (kip-ft/ft)

Distance from centerline box girder (ft)

HL-93 truck only—maximum values
Truck only—minimum values
Truck & lane—maximum values
HL-93 truck & lane—minimum values
HS25 only—maximum values
HS25 only—minimum values

FIGURE 3.51 Transverse deck live load moment envelope (unfactored without impact).
calculated. Using the selected tendon forces per unit length, the size and spacing of transverse tendons in the segment are determined.

The LRFD Strength Limit States may be also tabulated in a spreadsheet and an envelope of maximum and minimum values is determined for each chosen section. The values of the moment envelope can then be compared to the calculated bending capacities for each of the corresponding transverse components. Concrete service stresses were computed using several AASHTO Standard Specifications and the current LRFD Specifications for comparison purposes (see Figure 3.53).
3.5.5 Service Limit State Design

When checking concrete stresses, only Service Limit State I is checked with a live load factor of 1.0 for tension as well as compression. Also, a linear temperature gradient of 10° F between interior and exterior surfaces of the box is used in Service Limit State I. The current LRFD specification does not specify this loading, leaving it up to the owner or designer to establish if it should be included on a project-by-project basis. This example is based on a load factor of 0.5 for transverse temperature gradient when accompanying live load. Also, in addition to Service Limit State I, LRFD requires a check for service load stresses due to dead load and full temperature gradient. This limit state can often govern at locations where live load influences are small.

In addition to service limit states under maximum loading, temporary stresses such as those prior to barrier placement and vehicular traffic should be checked to ensure that allowable stresses are not exceeded during the construction process.

Service load combinations used in this example include the following:

Service I (Tension & Compression)

\[ 1.0(D + DW + EL) + 1.0(PT) + 1.0(LL + IM) + 1.0(CR + SH) + /- 0.5(TG) \]

Segmental Load Combination (LRFD Equation 3.4.1-2)

\[ 1.0(D + DW + EL) + 1.0(PT) + 1.0(CR + SH) + /- 1.0(TG) \]

3.5.6 Strength Limit State Design

3.5.6.1 Flexural Strength Design Check

For purposes of the transverse design, Strength Limit State IV is the same as Strength Limit State I without live load, with 25% more self-weight. This loading does not govern in this example.

For temperature gradient load factors, LRFD Specifications suggest determining a load factor on a project-specific basis, with a recommendation of 0.0 for most instances. Since these loads are a result of restrained deformations, the loads should disappear if the reinforcement begins to yield at ultimate. In addition, the Segmental Guide Specifications do not include this component in ultimate load combinations. For these reasons, the temperature gradient was not used in the strength limit state combinations below.

The LRFD specifications require minimum reinforcement equal to that required to resist 1.2 times the cracking moment. This requirement governs only the bottom slab (soffit) design. To satisfy the minimum steel requirement, the transverse bar spacing in the bottom soffit was decreased from 12 (205) to 8 in (203 mm), which represents an increase in reinforcement of 50%.

Also, under ultimate flexure, the amount of web steel reinforcing required for transverse bending should be calculated. This should be combined in an appropriate manner with reinforcing required for longitudinal shear.

Listed below is the ultimate load combination per LRFD:

Strength I

\[ \gamma_fDC + \gamma_fDW + 1.0EL + 1.75(LL + IM) + 0.5(CR + SH) \]

3.5.6.2 Shear Strength Design Check

Traditionally, shear behavior has been ignored in the design of concrete decks for AASHTO bridges. Box girder decks are similar in this sense, but can often have large construction loads placed on them. In these special cases, both one-way and two-way action shear should be investigated.
3.6 Longitudinal Design

3.6.1 Design Methodology

The following design example illustrates the longitudinal design process for a typical precast segmental bridge. The structure is assumed to be erected using the precast balanced cantilever construction method with a ground-based crane. Owing to changes in the statical system during erection, as cantilevers are made continuous through cast-in-place closure joints, it is necessary to analyze the structure for time-dependent effects. Time-dependent analysis is a function of the segment casting date, times that the segments are incorporated into the structure, as well as dates associated with changes in the statical system throughout the construction process.

It is customary to establish an assumed sequence of construction and to estimate a reasonable construction schedule. Casting and erection dates of the segments are established on the basis of the construction schedule and the segment production rate. Casting dates are a function of an assumed number of casting cells and the time required to cast each segment. For purposes of estimating these dates, the segment production rate is assumed as one typical segment per day per casting cell and one pier/expansion joint segment per week per casting cell. Segments are not to be erected earlier than one month after casting. During construction, when actual casting and erection dates become available, the stage-by-stage analysis should be rerun in order to obtain correct camber values.

Time-dependent properties of concrete are established on the basis of the environmental humidity and dimensions of the cross section, and can be adjusted for concrete composition (e.g. limestone aggregate), rate of hardening, and ambient temperature. Section properties are determined for each segment considering the effects of shear lag in the top and bottom slab.

The above information is entered into a time-dependent analysis proprietary software. A stage-by-stage analysis is performed using an assumed posttensioning layout while carefully modeling appropriate boundary conditions for each step of the construction process. After the construction has been modeled, the structure is stepped through time day 10,000 to allow all time-dependent effects to occur. It is also essential in statically indeterminate structures to sum up all locked-in forces that result from various stages of structural systems until day 10,000. Additional loads are placed on the structure such as live load, temperature gradient, and support settlement, as appropriate, and analyzed for initial (at end of construction) and final conditions at day 10,000.

3.6.2 Tendon Layout and Envelope

An approximate tendon layout can be based on preliminary calculations for construction loading of a typical cantilever. Span continuity tendons can be estimated by preliminary design based on the final structure approximate creep and shrinkage effects using load factor dead and live load combinations. The assumed layout can then easily be modified during the final design to satisfy all applicable LRFD Limit State Load Combinations.

The preliminary design for this example indicates the need for twelve cantilever tendons and five bottom continuity tendons per web. On the basis of previous experience, two eight-strand continuity tendons were added in the top slab across the closure pour to control stresses resulting from temperature gradients. The final design resulted in an increase of one cantilever tendon and one bottom span continuity tendon at interior spans only (see Figures 3.54 through 3.56).

The tendons used are based on a 12-strand system using 0.6 in (15.24 mm) diameter strands. Only 11 strands were used for bottom continuity tendons to provide space for 5% contingency posttensioning as required for internal tendons. One out of 12 strands will provide approximately 8% of the contingency posttensioning if needed. An empty duct was provided for the cantilever tendons combined with an anchorage on the last segment of the cantilever to allow for contingency posttensioning. This empty duct should be grouted if no contingency tendons are required.
Provisions are also made for future posttensioning by addition of anchorages and deviation points for external tendons (inside the box section), which can be used for adjustment of deflections or for other unforeseen conditions. Provisional posttensioning ducts and anchorages are covered under Article 5.14.2.3.8 of AASHTO LRFD Bridge Design Specifications (AASHTO 2012).

### 3.6.3 LRFD Live Load

LRFD live load (HL-93) consists of a single design truck per lane or tandem combined with a uniformly distributed lane load. For negative moments only, a second truck is added and the total effect is reduced by 10%. The second truck is required only between points of uniform load contraflexure, and should leave a space of at least 50 ft (15 m) between trucks measured between the rear axle of...
FIGURE 3.56 Typical tendon layout to balanced cantilever bridge construction.
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FIGURE 3.56 (Continued) Typical tendon layout to balanced cantilever bridge construction.
the leading truck and the front axle of the trailing truck. A fatigue truck is also specified but was not considered for this example.

A dynamic load allowance (impact) of 33% is added to the design truck, but is not required for design lane load. Multiple presence factors range from 1.2 for a single lane to 0.85 for three lanes and 0.65 for more than three lanes. This example is based on three lanes, and has a multiple presence factor of 0.85. The application of LRFD live loads is shown in Figures 3.57 and 3.58.

FIGURE 3.57 Application of LRFD live loads on a continuous structure.
3.6.4 Shear Lag Effect

The AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges, First Edition (AASHTO 1989a) adopted shear lag provisions of DIN 1075 (German Concrete Code) using a linear transition of effective flanges. However, in the second edition, shear lag provision changed to a step function between span and support regions. In contrast to this change, the AASHTO LRFD Bridge Design Specifications, Third Edition (AASHTO 2004) adopted shear lag provisions similar to DIN 1075, as shown in Article 4.6.2.6.2. The difference between the two methods is insignificant, but the LRFD shear lag provision is considered to be more accurate.

When determining section properties, it is commonly assumed that shear lag applies to the moment of inertia and location of the neutral axis of the section. However, the cross-sectional area remains based on the full gross cross section, so as to not overestimate the “P/A” component of posttensioning stresses, where \( P = PT \) effective force and \( A \) = gross cross section area.

Shear lag is a function of the structural system at the time it is under consideration. If the software used permits, section properties can be changed in the construction model to approximate true statical conditions at all intermediate steps. This additional accuracy may not be warranted for all designs, but could be evaluated on a case-by-case basis.

The following shear lag effect calculation is in accordance with article 4.6.2.6 of AASHTO LRFD Bridge Design Specifications (AASHTO 2012).

A. Completed structure
A.1- End span (see Figure 3.59)
where

\[ b = \text{flange width on each side of web (See Figure 3.60)} \]
\[ b_1 = 10.37’ \]
\[ b_2 = 9.71’ \]
FIGURE 3.59 End span effective flange width diagram.

Typical section  Support section

FIGURE 3.60 Typical box girder section effective flange widths.

\[ b_3 = 7.34' \]

\[ a = \text{the largest of } b, \text{ but not exceeding } 0.25 \times l \]

\[ = 10.37' < 0.25 \times 150' = 37.5' \]

\[ c = 0.1 \times l = 0.1 (150') = 15' 0'' \]

\[ l_i = 0.8 \times l = 0.8 (150') = 120' \]

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<tr>
<td>( b )</td>
<td>( b/l_i )</td>
<td>( b_s/b )</td>
<td>( b_m/b )</td>
<td>( b_{se} )</td>
</tr>
<tr>
<td>( b_1 )</td>
<td>10.37'</td>
<td>0.086</td>
<td>0.8</td>
<td>1.0</td>
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<tr>
<td>( b_2 )</td>
<td>9.71'</td>
<td>0.081</td>
<td>0.8</td>
<td>1.0</td>
</tr>
<tr>
<td>( b_3 )</td>
<td>7.34'</td>
<td>0.061</td>
<td>1.0</td>
<td>1.00</td>
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Obtained \( b_s/b \) and \( b_{me}/b \) ratios from LRFD Figure 4.6.2.6.2-2.

Effective flange: \( b_m \) (no reduction)

\[ b_{sle} = 8.3' \]

\[ b_{sls} = 7.77' \]

\[ b_{sle} = 7.34' \] (no reduction)

A.2-Inner span (see Figure 3.61)

where

\[ c = 0.1 \times l = 0.1 \times 200' = 20' \]

\[ l_i = 0.6 \times l = 0.6 \times 200' = 120' \]

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<td>( b_2 )</td>
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<td>0.080</td>
<td>0.8</td>
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<tr>
<td>( b_3 )</td>
<td>7.34'</td>
<td>0.060</td>
<td>1.0</td>
<td>1.00</td>
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Effective flange: $b_{me}$ (no reduction)
\[
\begin{align*}
  b_{s1e} &= 8.3' \\
  b_{s2e} &= 7.77' \\
  b_{s3e} &= 7.34' \text{ (no reduction)}
\end{align*}
\]

B. During construction

B.1- Cantilever (see Figure 3.62)

where $l_i = 1.5 \times l = 1.5 \times 98.75 = 148.125'$

Effective flange: $b_{sle} = 7.77'$
\[
\begin{align*}
  b_{sle} &= 7.28' \\
  b_{sle} &= 7.34' \text{ (no reduction)}
\end{align*}
\]

### 3.6.5 Temperature Load

Temperature loads for superstructures consist of uniform temperature change as well as temperature gradients. A uniform temperature change of the superstructure is defined as the entire cross-section heating or cooling at the same rate. In contrast to this, a temperature gradient is defined as a vertical temperature change from top to bottom of the box. A positive temperature gradient results from solar heating of the deck surface and will cause higher temperatures in the top deck. A negative temperature gradient results from rapid cooling of deck concrete while ground temperatures may remain relatively unchanged from daytime conditions. The aforementioned gradients vary in a nonlinear manner with respect to depth of the superstructure, which requires a rather complex method of analysis to determine resulting stresses. The AASHTO LRFD Bridge Design Specifications (Article 3.12.3) adopted a temperature gradient profile (see Figure 3.63) that differs from that used by the AASHTO Guide Specifications for Thermal Effects in Concrete Bridge Superstructures (AASHTO 1989b), which is an abridged version of NCHRP Report 276.
Both uniform temperature and temperature gradient are included in service limit state load combinations. Temperature gradient may be reduced by 50% if live load is present in service load combinations. For segmental bridge design only, a special load combination (LRFD Equation 3.4.1-2) for service shall be checked. This load combination has no live load; therefore 100% of the temperature gradient is included. In general, this load combination controls for segmental concrete bridges where live load force effects are small. In this example, such an area occurs at closure pours in the top of the box. Please note, for uniform temperature use a load factor of 1.0 when checking stresses, and 1.2 for structural deformations. The 1.2 factor will assure that bearing and expansion joint are not under design.

Temperature gradient is not included in strength limit state load combinations, while uniform temperature is included. Two load factors are assigned to uniform temperature in strength limit states. A factor of 0.5 is used for strength capacity calculations and 1.2 for structural deformations.

### 3.6.6 Time-Dependent Effect

Creep and shrinkage of concrete, including relaxation of prestressing steel are commonly referred to as **time-dependent long-term effects**. These effects are important factors that demand consideration in design of segmental bridges (see Figure 3.64). Nonlinear time-dependent deformations result in force redistribution due to changes in the statical system during the course of the construction, and continue through day 10,000 when long-term effects are considered to be diminished (see Figure 3.65).

Shrinkage, which causes shortening of concrete due to dehydration, is independent of stress (applied loads). Creep is a result of concrete deformation under permanent stress (loads) in addition to elastic deformation. The redistribution of sectional forces due to change in statical system and creep effect can be estimated by Dischinger’s equation:

$$M_f = M_i + (M_i - M_R) e^{-\phi}$$

where

- $M_f$ = Final moment at day 10,000
- $M_i$ = Moment as constructed at the end of construction
**FIGURE 3.64** Superimposed final moment (diagram 318), moment (diagram 301) and moment constructed on false-work (diagram 303).

**FIGURE 3.65** Development of time-dependent bending moment due to creep and shrinkage from end of construction (diagram 312) to day 10,000 (diagram 310).
\[ M_{II} = \text{Moment assuming the bridge is constructed on false work} \]
\[ \phi = \text{Creep coefficient} \]
\[ M_{cr} = \text{Moment due to creep effect} \]

The above equation can be rewritten to obtain \( M \) due to creep effects: \( M_{cr} = (1-e^{-\phi}) (M_{II} - M_I) \).

Steel relaxation is the loss of tension in prestressing steel under constant length and temperature over a period of time. To prevent excessive relaxation loss in segmental bridges, low-relaxation strands are used. The low-relaxation strands meet the ASTM Standard requirement that relaxation loss after 1000 hours under 70°F is no more than 2.5% when initially stressed to 70% guaranteed ultimate tensile strength (GUTS) and not more than 3.5% when stressed to 80% GUTS.

Although AASHTO LRFD Bridge Design Specifications allow creep and shrinkage effects to be evaluated using the provisions of CEB-FIP Model Code or ACI 209, for segmental bridge design, the CEB-FIP Model Code provisions are commonly used. This design example utilizes the CEB-FIP Model Code 1990.

### 3.6.7 Secondary Forces

Secondary forces are internal forces generated as a result of applied deformations or imposed loads to statically indeterminate systems.

Several recognized secondary forces in segmental bridge design are as follows:

- Secondary forces due to primary posttensioning (see Figures 3.66 and 3.67)
- Secondary forces due to construction process such as locked-in forces (see Figures 3.66 and 3.67)
- Secondary forces due to creep and shrinkage effects
- Secondary forces due to temperature loads (uniform and gradient temperature)
- Secondary forces due to support settlement

All of the above secondary forces are included in service limit state load combinations without exception. However, inclusion of different types of secondary forces in strength limit state load combinations may differ from code to code.

![FIGURE 3.66 Superimposed of PT primary moment (292) and secondary moment and locked-in forces (290) at day 10,000.](image-url)
Under *AASHTO LRFD 6th Edition* (2012), the locked-in forces (EL) are separated from prestressing secondary forces (PS), unlike in the previous editions. For strength limit state load combinations, both EL and PS have a load factor of 1.0, while dead loads (DC) have a load factor of 0.9 (minimum) and 1.2 (maximum). In most segmental bridge software, dead loads are not distinguished from locked-in forces. Owing to many construction stages during the erection process, it is possible to accumulate large quantities of dead load cases and locked-in force load cases. Once the construction process is complete, back-tracking to separate dead load cases from locked-in load cases creates complex book-keeping, and serves of little benefit to end results. EL load factor should follow DC, since it modifies the final design forces due to many construction stages, unlike other types of structures such as steel.

Secondary forces due to temperature gradient are not included in strength limit state load combinations, while support settlement secondary forces are to be considered on a project-specific basis. Uniform-temperature secondary forces, including creep and shrinkage effects, are included in strength limit state load combinations with a load factor of 0.5.

### 3.6.8 Summary of Design Forces

The summary of all design forces are presented in the form of maximum design forces envelopes as shown in Figures 3.68 and 3.69.

### 3.6.9 Service Limit State Design

Service limit state design of the superstructure requires a stress check for three load combinations. These consist of Service Limit State I, Service Limit State III, and a special load case for segmental bridges. Service Limit State III allows tension to be evaluated using a 0.8 live load factor, while Service Limit State I checks compression with a 1.0 live load factor. In combination with these three limit states, a nonlinear temperature gradient will be applied. For Service Limit States I and III, which use maximum live load influence, LRFD recommends a factor of 0.5 for temperature gradient in lieu of project-specific data. For the special load case applying to segmental bridges, temperature gradient receives a load factor of 1.0, since live load is not included. For a description of this load case, see LRFD Equation 3.4.1-2.
It is important to note that, although the special load case may not control at locations where large amounts of posttensioning are present, it may indeed control at locations where live load effects are small or at locations outside the precompressed tensile zone. Such locations for this example include tension in the top of closure pours and compression in the top of the box at pier locations. For this example, tendons were added at the top of the box crossing the closure pour to counteract the tension produced by the bottom of the box being warmer than the top.
3.6.10 Principal Tension Stress Check

In order to control diagonal tension cracks from developing in the webs adjacent to pier support at service limit state load combinations for shear and torsion, principal tension stresses should be checked. Stresses are calculated using Mohr’s circle to determine the principle tension (see Figure 3.70). If the allowable tensile capacity of the concrete is exceeded, diagonal tension cracks may be anticipated. Typically, the maximum principal tension stress is limited from $3\sqrt{f'_c}$ to $4\sqrt{f'_c}$ (psi). AASHTO LRFD limits the principal tension stresses to a maximum value of $3.5\sqrt{f'_c}$ (psi) at service loads (Tables 5.9.4.1.2-1 and 5.9.4.2.2-1) and $4\sqrt{f'_c}$ (psi) during construction (Table 4.14.2.3.3-1) for segmental bridges. Although this check is only required at the neutral axis of the web, it is recommended that the top slab and web interface location be investigated as well. For this example, $3.5\sqrt{f'_c}$ tension is used as a maximum allowable value under service loading.

Since principal stress is a function of longitudinal, vertical, and shear stress, it is necessary to determine concurrent moments for the maximum live load shear. It should be noted that high principle stresses commonly occur at interior pier locations, and the HL-93 live load moment corresponding to shear should only use one truck, rather than two, as used in calculating the negative moment at interior piers. The live load also has a load factor of 0.8 similar to Service III Limit State or it would be practically impossible to satisfy principal stresses while the extreme fiber could be in tension.

The maximum principal stresses in this example occurred near the interior piers at the top of the web for final conditions. From analysis at the critical section, the maximum principle tension stress was approximately $4.5\sqrt{f'_c}$; larger than the previously discussed limit. For this particular example, vertical posttensioning bars are used to control the principal tension stress. Calculations show that three 1¼” diameter bars, are needed in each web to the reduce principle tension to an acceptable value (see Figure 3.71). The overstress could also be addressed by modifying the cross section (web thickness) or adding more longitudinal compressive stress (additional strands). The solution presented was deemed acceptable since only a small number of segments require vertical posttensioning. A graph of the principle stress prior to addition of vertical posttensioned bars can be seen in Figure 3.72.

Principal tensile stress check

$$\nu = \frac{VQ}{Ib}$$

where

$V$ = Vertical shear force
$Q$ = First moment of an area with respect to CG of the section
$I$ = Moment of inertia about CG of the section
$b$ = Perpendicular web thickness

$$f_1 = \frac{\sigma_x + \sigma_y}{2} - \frac{1}{2} \sqrt{4\nu^2 + (\sigma_x - \sigma_y)^2}$$

FIGURE 3.70 Principal stresses and Mohr’s Circle.
where compression stress is positive

For $\sigma_y = 0$: (at sections where no vertical web posttensioning is present)

$$v_a = \sqrt{f_a \times (f_a + f)}$$

where

$f_a = $ Allowable principal tension
$f = $ Compressive stress at level on web under investigation

**FIGURE 3.71** Vertical PT bar in the web.

**FIGURE 3.72** Principal tension stress at service limit state load combinations along the bridge.
3.6.11 Flexural Strength Check

Once service stresses are satisfied in the superstructure, the limit state of flexural strength must be checked. For most cases with superstructures, Strength Limit State I is the only load combination that needs to be considered. However, for longer spans where the ratio of the dead load to live load is large, Strength Limit State IV may control. For this example, the magnitudes of live load force effects are greater than a 25% difference in structural component dead load. Hence, Strength Limit State IV will not control.

The load factors for support settlement and temperature gradient are not provided by LRFD; they are to be determined on a project-specific basis. In lieu of project specific data, LRFD recommends using a load factor of 0.0 for the temperature gradient. With regard to the temperature gradient, the loads imposed result from restrained deformations and should disappear if the reinforcement starts to yield at ultimate. Owing to this occurrence, the temperature gradient is not considered in strength limit states. Also, support settlements are not considered in this example.

The LRFD specifications, Article 5.7.3.3.2, require minimum reinforcement in order to prevent flexure brittle failure, especially in deeper girder where a very small amount of flexural reinforcement is required.

3.6.12 Shear and Torsion Design

3.6.12.1 AASHTO LRFD Shear and Torsion General Design Procedure

The modified compression field theory (MCFT) was developed by Collins and Mitchell (Collins and Mitchell, 1991) in Canada. The MCFT for shear and torsion design was adopted for the first time by the Ontario Highway Bridge Design Code in 1991. The AASHTO LRFD Bridge Design Specifications (AASHTO 2012) also adopted the new method of shear and torsion design in addition to traditional ACI and AASHTO Guide Specifications for segmental bridge empirical equations. Unlike previous empirical equations, MCFT is a rational approach that gives physical significance to the parameters being calculated. The MCFT is based on variable-angle truss instead of a 45° truss model. Owing to this truss model, the longitudinal reinforcement becomes an important element of shear design. Prior to 2008 AASHTO LRFD Interim, the general procedure for concrete shear capacity contribution required iteration to determine $\alpha$ and $\theta$ parameters. Currently, AASHTO provides noniterative algebraic expressions. The iterative procedure has been moved to Appendix B5 and it is acceptable as an alternative to General Procedure Section 5.8.3.4.2. AASHTO LRFD also allows $V_c$ to be calculated using a simplified procedure as per Section 5.8.3.4.3 compatible with ACI Code 318-05 and AASHTO Standard Specification. For segmental box girder bridges, shear and torsion design procedure of Section 5.8.6 may be applied in lieu of Section 5.8.3.

3.6.12.1.1 Sections Subjected to Shear Only

In a box girder, the stresses due to shear and torsion are additive on one side of the web and counteract each other on the other side. Therefore, the final transverse web reinforcement should be based on the summation of reinforcement due to shear and torsion.

Normally, the loading that produces the maximum shear is not the same loading which produces the maximum torsion. Therefore, it is conservative to design on the basis of the maximum shear and maximum torsion. However, it is sufficient to design using the maximum shear with its associated torsion and the maximum torsion with its associated shear.

For shear design, the following basic relationship must be satisfied at each section:

$$V_u \leq \phi V_n$$

where

$$V_n$$ is determined as the lesser of

$$V_n = V_c + V_t + V_p$$  \hfill (LRFD 5.8.3.3-1)
The value of β at a given section is obtained from equation:

$$\beta = \frac{4.8}{1 + 750\varepsilon_s}$$  \hspace{1cm} (LRFD 5.8.3.4.2-1)

where β is a factor indicating the ability of diagonally cracked concrete to transmit tension and shear, and ε_s is the net longitudinal tensile strain in the section at the centroid of the tension reinforcement. Tensile strain ε_s may be determined by equation:

$$\varepsilon_s = \frac{M_u/d_s}{E_sA_s + E_pA_{ps}} + 0.5N_u + |V_u - V_p| - A_{ps}f_{ps}$$  \hspace{1cm} (LRFD 5.8.3.4.2-4)

The contribution of transverse reinforcement is calculated using equation:

$$V_s = \frac{A_s f_s d_s (\cot \theta + \cot \alpha) \sin \alpha}{s}$$  \hspace{1cm} (LRFD 5.8.3.3-4)

where α is the angle of inclination of transverse reinforcement and θ is the angle of inclination of diagonal compressive stresses as determined by equation:

$$\theta = 29 + 3500\varepsilon_s$$  \hspace{1cm} (LRFD 5.8.3.4.2-3)

### 3.6.12.1.2 Longitudinal Reinforcement

One of the cornerstone principles of modified compression field theory is the recognition that shear causes tension in longitudinal steel. At each section of the beam not subjected to torsion, the capacity of the longitudinal reinforcement must be checked for sufficiency. This relationship is expressed as follows:

$$A_s f_s + A_{ps} f_{ps} \geq \left[ \frac{M_u}{d_s \phi_f} + 0.5 \frac{N_u}{\phi_c} + \left( \frac{V_u}{\phi_v} - V_p - 0.5V_s \right) \cot \theta \right]$$  \hspace{1cm} (LRFD 5.8.3.5-1)

### 3.6.12.1.3 Sections Subjected to Combined Shear and Torsion

For sections subjected to combined shear and torsion, refer Article 5.8.3.6. Strain should be calculated taking into account the combination of these effects. Shear stress, longitudinal reinforcing, and area of shear reinforcing should also be modified.

### 3.6.12.2 AASHTO Shear and Torsion Design Procedure for Segmental Box Girder Bridges

Refer to LRFD Article 5.8.6

The nominal shear resistance is the algebraic sum of the contributing components:

$$V_n = V_c + V_s + V_p$$
In sections where the effects of torsion may be neglected, the nominal shear resistance is limited to

\[ V_n = 0.379 \sqrt{f'_c b_y d} \quad \text{(LRFD 5.8.6.5-2)} \]

The shear provided by concrete section is determined as follows:

\[ V_t = 0.0632 K \sqrt{f'_c b_y d} \quad \text{(LRFD 5.8.6.5-3)} \]

where \( K \) is the stress variable parameter expressed by equation:

\[ K = \sqrt{1 + \frac{f_{pc}}{0.0632 \sqrt{f'_c}}} \leq 2.0 \quad \text{(LRFD 5.8.6.3-3)} \]

where

- \( f_{pc} = \) Compressive stress in concrete after allowance for all prestress losses at the centroid of cross-section resisting shear
- \( f'_c = \) Specified concrete strength

Shear provided by transverse reinforcement is determined from equation:

\[ V_t = \frac{A_v f_s d}{s} \quad \text{(LRFD 5.8.6.5-4)} \]

\( A_v = \) Area of transverse reinforcement within a distance \( s \) (in²)

Torsional effects is investigated in sections where factored torsional moment at the section is greater than the torsional cracking moment:

\[ T_u \geq \phi T_{cr} \quad \text{(LRFD 5.8.6.3-1)} \]

where

- \( T_u = \) Factored torsional moment (kip-ft)
- \( T_{cr} = \) Torsional cracking moment (kip-ft)

Torsional cracking moment \( T_{cr} \) is given by equation:

\[ T_{cr} = 0.0632 K \sqrt{f'_c} 2 A_o b_e \quad \text{(LRFD 5.8.6.3-2)} \]

where

- \( A_o = \) Area enclosed by shear flow path (in²)
- \( b_e = \) Minimum effective shear flow web or flange width to resist torsional stresses (in)

When torsional effects are considered, the longitudinal and transverse reinforcement must satisfy the following conditions:

\[ T_u \leq \phi T_n \quad \text{(LRFD 5.8.6.4-1)} \]

where \( T_n \) is the nominal torsional resistance of transverse reinforcement calculated as follows:

\[ T_n = \frac{2 A_o A_v f_s}{s} \quad \text{(LRFD 5.8.6.4-2)} \]
The longitudinal reinforcement should satisfy:

\[ A_i = \frac{T_u p_b}{2A_v f_y} \]  

(LRFD 5.8.6.4-3)

where

\[ A_i = \text{Total additional longitudinal reinforcement required for torsion (in}^2) \]
\[ p_b = \text{Perimeter of centerline outermost continuous closed transverse reinforcement (in)} \]

Additionally the section is sized to satisfy:

\[ \left( \frac{V_n}{b_s d_v} \right) + \left( \frac{T_u}{2A_v b_t} \right) \leq 0.474 \sqrt{f'_c} \]  

(LRFD 5.8.6.5-5)

### 3.6.12.2.1 Design Examples (Using LRFD Modified Compression Field Theory)

#### 3.6.12.2.1.1 Node Number: 41 (At Critical Shear Section)

Ultimate moment: \( M_u = 82,091 \text{ kip-ft} \) (negative moment, bottom slab is in compression)

\[ V_u = 2,391 \text{ kip} \]

\( \phi = 0.90 \) for shear

Nominal shear resistance:

\[ V_n = V_c + V_v + V_p \]

or

\[ V_n = 0.25 f'_c b_s d_v + V_p \]

where

\( f'_c = 6 \text{ksi, compression strength of concrete} \)
\( b_s = 32 \text{ in, effective web width} \)
\( d_v = 108 - 6 - 17.1/2 = 93.4 \text{ in} = 7.79 \text{ ft} > \text{Max} \left\{ 0.9(108 - 6), 0.72 \times 108 \right\} \)

effective shear depth

\[ V_p = 0 \]

\[ V_n = 0.25 \times 6 \times 32 \times 93.4 = 4,483 \text{ kip} \]

\[ V_n = 4,483 \text{ kip} > V_u / \phi = 2,391 / 0.9 = 2,657 \text{ kip} \]

Cross-section dimension is sufficient.

Concrete contribution:

\[ V_c = 0.0316 \beta \sqrt{f'_c b_s d_v} \]

Transverse reinforcement contribution:

\[ V_v = \frac{A_v f_v d_v \cot \theta}{s} \]
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where

\[ \beta = \text{factor indicating the ability of diagonally cracked concrete to transmit tension} \]
\[ \theta = \text{angle of inclination of diagonal compressive stresses} \]

Step 1: Calculate the net longitudinal tensile strain in the section at the centroid of the tension reinforcement:

\[
\varepsilon_t = \frac{M_u + 0.5N_u + |V_u - V_p| - A_{ps}f_{ps}}{E_A + E_pA_{ps}} = \frac{82,091}{7.79} + \frac{2,391 - 67.7 \times 189}{28,500 \times 67.7} = \frac{133.7}{1,929,450} = 0.00007
\]

Step 2: Compute the value of \( \beta \):

\[
\beta = \frac{4.8}{1 + 750\varepsilon_t} = \frac{4.8}{1 + 750 \times 0.00007} = 4.53
\]

Step 3: Compute the value of \( \theta \):

\[
\theta = 29 + 3,500\varepsilon_t = 29 + 3,500 \times 0.00007 = 29.24^\circ
\]

Step 4: Compute the contribution of concrete steel \( V_c \):

\[
V_c = 0.0316\beta \sqrt{f_y}b_d
\]

\[
= 0.0316 \times 4.53 \times \sqrt{6} \times 32 \times 93.4 = 1,048 \text{ kip}
\]

\[
V_t = V_u - V_c = V_u / \phi - V_c
\]

\[
= 2,391/0.9 - 1,048 = 1,609 \text{ kip/web}
\]

\[
A_{ps} = \frac{V_t}{f_yd_v \cot \theta}
\]

\[
= \frac{805}{60 \times 93.4 \times \cot 29.24^\circ} = 0.08 \text{ in}^2/\text{in} = 0.96 \text{ in}^2/\text{ft}
\]

Use double #6 bars at 9” centers per web \( A_v = 1.17 \) in²/ft

3.6.12.2.1.2 Longitudinal Reinforcement

For sections not subjected to torsion, longitudinal reinforcement needs to satisfy:

\[
A_{sfy} + A_{ps}f_{ps} \geq \left[ \frac{M_u}{d\phi} + 0.5 \frac{N_u}{\phi} + \left( \frac{V_u}{\phi} - 0.5V_t - V_p \right) \cot \theta \right]
\] (LRFD 5.8.3.5-1)

\( \phi = 0.95 \) for flexure; (Table 5.5.4.2.2-1)
\( \phi = 0.90 \) for shear; (Table 5.5.4.2.2-1)

\[
A_{sfy} + A_{ps}f_{ps} = 0 \times 0 + 67.7 \times 253 = 17,136 \text{ kip}
\]
Therefore, the condition (5.8.3.5-1) is satisfied.

3.6.12.2.1.3 Node Number: 29 (At Section 60 Feet from the Face of Diaphragm)        Ultimate moment:  

\[ M_u = 20,816 \text{ kip-ft} \] (positive moment, top slab is in compression)

\[ V_u = 1,087 \text{ kip} \]

\[ \phi = 0.90 \]

Nominal shear resistance:

\[ V_n = V_c + V_s + V_p \]

or

\[ V_n = 0.25 f'_c b_c d_s + V_p \]

where

\[ f'_c = 6 \text{ ksi}, \] compression strength of concrete

\[ b_c = 32 \text{ in}, \] effective web width

\[ d_s = 108 - 5 - 2.6/2 = 101.7 \text{ in} = 8.48 \text{ ft} > \text{Max}\{0.9(108 - 5), 0.72 \times 108\}, \text{effective shear depth}; \]

\[ V_p = 0 \]

\[ V_n = 0.25 \times 6 \times 32 \times 101.7 = 4,882 \text{ kip} \]

\[ V_n = 4,882 \text{ kip} > V_u/\phi = 1,087/0.9 = 1,208 \text{ kip} \]

Cross-section dimensions are sufficient

Concrete contribution:

\[ V_c = 0.0316 \beta \sqrt{f'_c b_c d_s} \]

Transverse reinforcement contribution:

\[ V_s = \frac{A_s f_s d_s \cot \theta}{s} \]
where

\[ \beta = \text{Factor indicating ability of diagonally cracked concrete to transmit tension} \]

\[ \theta = \text{Angle of inclination of diagonal compressive stresses} \]

Step 1: Calculate the net longitudinal tensile strain in the section at the centroid of the tension reinforcement:

\[
\varepsilon = \frac{M_{ud} + 0.5N_u + |V_u - V_p| - A_{ps}f_{ps}}{E_A + E_pA_p} = \frac{20,816}{8.48} + \frac{1,087 - 19.1 \times 189}{28,500 \times 19.1} = -68.2
\]

\[
\frac{544,350}{544,350} = -0.00013 \rightarrow \varepsilon = 0
\]

Step 2: Compute the value of \( \beta \):

\[
\beta = \frac{4.8}{1 + 750 \varepsilon} = \frac{4.8}{1 + 750 \times 0.0} = 4.8
\]

Step 3: Compute the value of \( \theta \):

\[
\theta = 29 + 3,500 \varepsilon = 29 + 3,500 \times 0.0 = 29.0^\circ
\]

Step 4: Compute the contribution of concrete steel \( V_c \):

\[
V_c = 0.0316 \beta \int_{f_y} b_y d_y = 0.0316 \times 4.8 \sqrt{6} \times 32 \times 101.7 = 1,209 \text{ kip}
\]

\[
V_u / \phi = 1,087 / 0.9 = 1,207 \text{kip} \cong V_c
\]

Minimum reinforcing \( A_v = 0.0316 \sqrt{f_y} b_y \frac{s}{f_y} = 0.0316 \sqrt{6} \times 16 \times 12 \times 60 = 0.248 \text{ in}^2/\text{ft} \)

Conservatively use double #5 at 18” centers \( A_v = 0.413 \text{ in}^2/\text{ft} \)

\[
V_i = \frac{A_v f_y d_v \cot \theta}{s} = \frac{2 \times 0.413 \times 60 \times 101.7 \times \cot 29^\circ}{12} \times 2 = 1,515.5 \text{ kip}
\]

3.6.12.2.1.4 Longitudinal Reinforcement For sections not subjected to torsion, longitudinal reinforcement needs to satisfy:

\[
A_v f_y + A_{ps} f_{ps} \geq \left[ \frac{M_{ud} + 0.5N_u}{d_A \phi} + \left( \frac{V_u}{\phi} - 0.5V_i - V_p \right) \cot \theta \right] \text{ (LRFD 5.8.3.5-1)}
\]

\( \phi = 0.95 \) for flexure; (Table 5.5.4.2.2-1)

\( \phi = 0.90 \) for shear; (Table 5.5.4.2.2-1)

\[
A_v f_y + A_{ps} f_{ps} = 0 \times 0 + 19.1 \times 268 = 5,118 \text{ kip}
\]
\[
\frac{M_u}{d_\theta \phi} + 0.5 \frac{N_u}{\phi} + \left( \frac{V_u}{\phi} - 0.5 V_f - V_p \right) \cot \theta \\
= \frac{20,816}{8.48 \times 0.95} + 0 + \left( \frac{1.087}{0.90} - 0.5 \times 1,515.5 - 0 \right) \times \cot 29^\circ \\
= 3,396 \text{ kip}
\]

Therefore, the condition (5.8.3.5-1) is satisfied.

3.6.12.2.2 Design Examples (Using AASHTO LRFD Section 5.8.6)

3.6.12.2.2.1 Node Number: 41 (At Critical Shear Section)

\[ V_u = 2,391 \text{ kip} \]

\[ \phi = 0.90 \text{ for shear} \]

\[ f_{pc} = 906 \text{ psi at neutral axis} \]

\[ f' = 6 \text{ ksi, compression strength of concrete} \]

\[ b_v = 32 \text{ in, effective web width} \]

Concrete contribution:

\[ V_c = 0.0632 K \sqrt{f'_c} b_v d \]

\[ K = \sqrt{1 + \frac{f_{pc}}{0.0632 \sqrt{f'_c}}} \leq 2.0 \]

\[ K = \sqrt{1 + \frac{0.9}{0.0632 \sqrt{6}}} = 2.62 \Rightarrow 2.0 \]

Note: Tensile stress at the extreme fiber under factored loads with effective prestressing was checked to insure it was under \( 6\sqrt{f'_c} \).

\[ V_c = 2 \times 0.0632 \sqrt{6} \times 32 \times 102 = 1,011 \text{ kip} \]

Transverse reinforcement contribution:

\[ V_t = V_n - V_c = \frac{V_u}{\phi} - V_c \]

\[ = 2,391/0.90 - 1,011 = 1,646 \text{ kip/web} \]

\[ A_s = \frac{V_t}{f_s d} \]

\[ = \frac{1,646}{60 \times 102} = 0.134 \text{ in}^2/\text{in} = 1.61 \text{ in}^2/\text{ft} \]

Use double #6 bar at 6” centers per web \( A_s = 1.76 \text{ in}^2/\text{ft} \)

\[ V_t = \frac{A_s f_s d}{s} \]

\[ V_t = \frac{2 \times 1.76 \times 60 \times 102}{12} = 1,795 \text{ kip} \]
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Ultimate shear resistance:

$$\phi V_u = \phi (V_c + V_s + V_p)$$

$$V_p = 0$$

$$\phi V_u = 0.9(1,011 + 1,795) = 2,525 \text{ kip} > V_u = 2,391 \text{ kip}$$

Check maximum nominal shear resistance:

$$V_n = V_c + V_s + V_p \leq 0.379 \sqrt{f'_c} b_v d_v$$

$$V_n = 1,011 + 1,795 = 2,806 \text{ kip}$$

$$0.379 \sqrt{f'_c} b_v d_v = 0.379 \times \sqrt{6} \times 32 \times 103 = 3,060 \text{ kip}$$

The section is adequate to carry the factored shear force.

3.6.12.2.2 Node Number: 29 (At Section 60 Feet from the Face of Diaphragm)

$$V_u = 1,087 \text{ kip}$$

$$\phi = 0.90 \text{ for shear}$$

$$f_{pc} = 533 \text{ psi at neutral axis}$$

$$f'_c = 6 \text{ ksi, compression strength of concrete}$$

$$b_v = 32 \text{ in, effective web width}$$

Concrete contribution:

$$V_c = 0.0632 K \sqrt{f'_c} b_v d_v$$

$$K = \sqrt{1 + \frac{f_{pc}}{0.0632 \sqrt{f'_c}}} \leq 2.0$$

$$K = \sqrt{1 + \frac{0.533}{0.0632 \sqrt{6}}} = 2.11 \Rightarrow 2.0$$

Note: Tensile stress at the extreme fiber under factored loads with effective prestressing was checked to insure it was under $$6\sqrt{f'_c}$$.

$$V_c = 2 \times 0.0632 \sqrt{6} \times 32 \times 103 = 1,021 \text{ kip}$$

Transverse reinforcement contribution:

$$V_s = V_n - V_c = V_u/\phi - V_c$$

$$= 1,087/0.90 - 1,021 = 187 \text{ kip} = 93 \text{ kip/web}$$
\[
\frac{A_v}{s} = \frac{V_t}{f_y d}
\]

\[
= \frac{93}{60 \times 103} = 0.015 \text{ in}^2 / \text{in} = 0.18 \text{ in}^2 / \text{ft}
\]

Minimum reinforcing

\[
A_v = \frac{50b_s s}{f_y} = \frac{50 \times 16 \times 12}{60,000} = 0.16 \text{ in}^2 / \text{ft}
\]

Minimum reinforcing does not control. However, conservatively use double #5 at 18 in centers.

\[
A_v = 0.413 \text{ in}^2 / \text{ft}
\]

\[
V_t = \frac{V_t f_y d}{s} = \frac{2 \times 0.413 \times 60 \times 103}{12} = 425 \text{ kip}
\]

Ultimate shear resistance:

\[
\phi V_n = \phi (V_t + V_s + V_p)
\]

\[
V_p = 0
\]

\[
\phi V_n = 0.9(1,021 + 425) = 1,301 \text{ kip} > V_n = 1,087 \text{ kip}
\]

Check maximum nominal shear resistance:

\[
V_n = V_t + V_s + V_p \leq 0.379 \sqrt{f_y^2 b_s d}
\]

\[
V_n = 1,021 + 425 = 1,446 \text{ kip}
\]

\[
0.379 \sqrt{f_y^2 b_s d} = 0.379 \times \sqrt{6 \times 32 \times 103} = 3,060 \text{ kip}
\]

The section is adequate to carry the factored shear force.

### 3.7 Construction Stage Analysis

#### 3.7.1 Stability during Construction

A stability analysis during construction is one of the design criteria for segmental bridge design. During construction of a segmental bridge, the boundary conditions are constantly changing from the beginning of construction to the end. At all times during construction, the structure and foundation must be in a stable state and have ample safety factors against material failure, overturning, and buckling. Stability analysis, therefore, becomes an important design issue due to the lower degree of redundancy and the load imbalance of the structure during this period.

A free cantilever structure is one example that requires a stability check during erection of a segment (see Figure 3.73). The longer the span length, the larger are the unbalanced loads. In many cases, temporary supports are required to handle the load imbalance during erection. In addition to balanced cantilever conditions, other partially completed structures may also need to be investigated.

It is important for the engineer to specify the design plans and the construction loads that were assumed during design associated with the construction method selected. It is common practice that at least one construction method be designed and shown in the plans. The limits of these loads...
and locations where loads are applied on the structure should also be shown. Additionally, the engineer’s construction schemes should be clearly stated, including approximate support reactions due to construction equipment. The stresses caused by critical construction loads and strengths of the members should also be checked. Figures 3.74 through 3.76 show critical construction loads for this example.

The stability analysis specifications were originally covered in Article 7.4 of the AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges, Second Edition (AASHTO 1999). Later, those specifications were adopted by the AASHTO LRFD Bridge Design Specifications, under Article 5.14.2.3. Table 3.1 shows service limit state load combinations during construction and its associated stress limit.

The following construction loads should be considered in a stability analysis:

- $DC =$ Weight of the supported structure (kip).
- $DIFF =$ Differential load: applicable only to balanced cantilever construction, taken as 2% of the dead load applied to one cantilever (kip).
- $DW =$ Superimposed dead load (kips or klf).
- $CLL =$ Distributed construction live load; taken as 0.01 ksf of deck area applied to one side of cantilever and 0.005 ksf on the other side.
- $CE =$ Specialized construction equipment, load from launching gantry, form-traveler, beam and winch, etc. (kip).
- $IE =$ Dynamic load from equipment; determined according to the type of machinery. (For gradual lifting, it may be taken as 10% of the lifting load.)
- $CLE =$ Longitudinal construction equipment loads (kip).
- $U =$ Segment unbalanced load (kip).
- $WS =$ Horizontal wind load on structure in accordance with the provisions of Section 3 (LRFD) (ksf).
FIGURE 3.74  Construction loads Case A and Case B.

FIGURE 3.75  Construction loads Case C and Case D.
Segmental Concrete Bridges

FIGURE 3.76 Construction loads Case E and Case F.

TABLE 3.1 Service Limit State Load Combinations during Construction

<table>
<thead>
<tr>
<th>Load Combinations</th>
<th>Allowable Flexural Tension Stress (ksi)</th>
<th>Allowable Principal Tension Stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a1 ( DC + \text{DIFF} + \text{CLL} + (CE + IE) )</td>
<td>0.19( \sqrt{f_c} )</td>
<td>0.11( \sqrt{f_c} )</td>
</tr>
<tr>
<td>a2 ( DC + \text{DIFF} + \text{CLL} + (CE + IE) + \text{OTHER LOADS} )</td>
<td>0.22( \sqrt{f_c} )</td>
<td>0.126( \sqrt{f_c} )</td>
</tr>
<tr>
<td>b1 ( DC + U + \text{CLL} + (CE + IE) )</td>
<td>0.19( \sqrt{f_c} )</td>
<td>0.11( \sqrt{f_c} )</td>
</tr>
<tr>
<td>b2 ( DC + U + \text{CLL} + (CE + IE) + \text{OTHER LOADS} )</td>
<td>0.22( \sqrt{f_c} )</td>
<td>0.126( \sqrt{f_c} )</td>
</tr>
<tr>
<td>c1 ( DC + \text{DIFF} + 0.7\text{WS} + 0.7\text{WUP} )</td>
<td>0.19( \sqrt{f_c} )</td>
<td>0.11( \sqrt{f_c} )</td>
</tr>
<tr>
<td>c2 ( DC + \text{DIFF} + 0.7\text{WS} + 0.7\text{WUP} + \text{OTHER LOADS} )</td>
<td>0.22( \sqrt{f_c} )</td>
<td>0.126( \sqrt{f_c} )</td>
</tr>
<tr>
<td>d1 ( DC + \text{DIFF} + \text{CLL} + CE + 0.7\text{WS} + \text{WUP} + 0.7\text{WE} )</td>
<td>0.19( \sqrt{f_c} )</td>
<td>0.11( \sqrt{f_c} )</td>
</tr>
<tr>
<td>d2 ( DC + \text{DIFF} + \text{CLL} + CE + 0.7\text{WS} + \text{WUP} + 0.7\text{WE} + \text{OTHER LOADS} )</td>
<td>0.22( \sqrt{f_c} )</td>
<td>0.126( \sqrt{f_c} )</td>
</tr>
<tr>
<td>e1 ( DC + U + \text{CLL} + (CE + IE) + 0.3\text{WS} + 0.3\text{WE} )</td>
<td>0.19( \sqrt{f_c} )</td>
<td>0.11( \sqrt{f_c} )</td>
</tr>
<tr>
<td>e2 ( DC + U + \text{CLL} + (CE + IE) + 0.3\text{WS} + 0.3\text{WE} + \text{OTHER LOADS} )</td>
<td>0.22( \sqrt{f_c} )</td>
<td>0.126( \sqrt{f_c} )</td>
</tr>
<tr>
<td>f1 ( DC + \text{CLL} + (CE + IE) + \text{CLE} + 0.3\text{WS} + 0.3\text{WE} )</td>
<td>0.19( \sqrt{f_c} )</td>
<td>0.11( \sqrt{f_c} )</td>
</tr>
<tr>
<td>f2 ( DC + \text{CLL} + (CE + IE) + \text{CLE} + 0.3\text{WS} + 0.3\text{WE} + \text{OTHER LOADS} )</td>
<td>0.22( \sqrt{f_c} )</td>
<td>0.126( \sqrt{f_c} )</td>
</tr>
</tbody>
</table>

Notes:
1. OTHER LOADS = CR + SH + TU + TG + EH + EV + ES + WA
2. Allowable compressive stress in concrete where \( f_c \) is the compressive strength at the time of load application.
3. d: equipment not working
e: normal erection
f: moving equipment
WE = Horizontal wind load on equipment taken as 0.1 ksf of exposed surface.
WUP = Wind uplift on cantilever taken as 0.005 ksf of deck area applied to one side only.
A = Static weight of precast segment being handled (kip).
AI = Dynamic response due to accidental release of precast segment taken as static load to be added to the dead load as 100% of load A (kip).
CR = Creep effects in accordance with Article 5.14.2.3.6 (LRFD).
SH = Shrinkage in accordance with Article 5.14.2.3.6 (LRFD).
T = Thermal loads; the sum of the effects due to uniform temperature variation (TU) and temperature gradients (TG).
WA = Water load and stream pressure.

Strength limit state load combinations (see Figure 3.77)

1. For maximum force effects:

\[ \Sigma \phi R_n = 1.1(DC + DIFF) + 1.3CE + A + AI \]  

\text{(LRFD 5.14.2.3.4a-1)}

\text{FIGURE 3.77}  \quad \text{Strength limit state construction load combinations.}
2. For minimum force effects:

\[ \Sigma \phi F_u = DC + CE + A + AI \]  

(LRFD 5.14.2.3.4a-2)

WS, WE and other loads were ignored in this analysis.

Stress Limits for \( f'_c = 6 \) ksi:

Compressive stress = \(-0.5 f'_c = -0.5 \times 6 = -3 \) ksi

Tensile stress = \( 0.19 \sqrt{f'_c} = 0.19 \sqrt{6} = 0.465 \) ksi

Since the design example has a 200 ft typical span, only one balanced cantilever structure will be considered in the stability analysis during construction.

The load combinations “a” to “f” as specified in the AASHTO LRFD Bridge Design Specifications Table 5.14.2.3.3-1 were computed.

The following construction loads were applied in the stability analysis.

\[ CLL1 = 0.005 \text{ ksf} \times 43 = 0.215 \text{ klf} \]

\[ CLL2 = 0.01 \text{ ksf} \times 43 = 0.43 \text{ klf} \]

\( CE = \) construction equipment such as stressing jack and stressing platform  
\( = 5 \) Kip.

\( CE + IE = 5 \times 1.1 = 5.5 \) kip

\( W_{up} = 0.005 \text{ ksf} \times 43 = 0.215 \text{ klf} \)

\( A = 78 \times 12 \times 0.155 = 145 \) kip

1. For maximum force effects:

\[ \Sigma \phi F_u = 1.1 \times (DC + DIFF) + 1.3 \times CE + A + AI \]

2. For minimum force effects:

\[ \Sigma \phi F_u = DC + CE + A + AI \]

where

\( A = \) static load of typical segment = 145 kip

\( CE = 5 \) kip.
Although calculations have not been shown in this example, of load cases a to f, strength limit state load combination e controls.

3.7.2 Erection Tendons

It is common practice in precast balance cantilever segmental bridges to use temporary or permanent posttensioning bars to attach the segment being erected to the previously erected segment. In the case of permanent erection PT bars, the posttensioned bars could be designed as part of the permanent cantilever tendons and stressed to full allowable jacking force. However, if reusable temporary posttensioned bars are utilized, the jacking force should be limited to approximately 50% of GUTS of the bars.

The epoxy resin is applied to the match cast faces of the joint between two segments before posttensioning bars are stressed. Purposes of the epoxy resin are as follows:

1. Lubrication to facilitate the proper alignment between segments.
2. Hardened epoxy provides a water-tight joint, preventing moisture, water, and chlorides from reaching the tendons.
3. Hardened epoxy helps distribute compressive stresses and shear stresses more uniformly.
4. Hardened epoxy prevents cementitious grout in the tendon duct from leaking out.

The application of epoxy is normally 1/16 in thick applied on both faces of match cast joints.

In accordance with the Article 5.14.2.4.2 of the LRFD Specifications for a Type A joint, the temporary posttensioning bars should be designed to provide a minimum stress of 0.03 ksi and an average stress of 0.04 ksi across the joint until the epoxy has cured. The intention of the stress limitation is to prevent uneven epoxy thickness across the match-cast joint, which could lead to systematic error in geometry control.

Essentially, there are two load cases that need to be considered when designing temporary posttensioning bars:

1. Dead load of the segment plus construction loads and temporary posttensioning bars, (see Figure 3.78). The erection PT bars should be stressed during the open time of the epoxy (approximately 45 to 60 min). The allowable joint stresses for this load case should conform to Article 5.14.2.4.2 of the LRFD specifications.
2. Case 1 plus permanent cantilever tendons. Normally, one or two hours after the open time of the epoxy is completed, the allowable joint stress is zero tension, preferably some compression.

![Figure 3.78: Construction loads during segment erection.](image-url)
3.7.2.1 Design of Erection PT Bars

Section Properties (use typical section: including shear lag effect)

\[ A_c = 78 \text{ ft}^2 \]

\[ A_{c,\text{eff}} = 70.38 \text{ ft}^2 \]

\[ I = 791.892 \text{ ft}^4 \]

\[ Y_s = 3.4 \rightarrow S_y = 232.89 \text{ ft}^3 \]

\[ Y_b = 5.6 \text{ ft} \rightarrow S_b = 141.40 \text{ ft}^3 \]

\[ CLL.2 = 0.01(43) = 0.43 \text{ plf} \]

Segment weight + DIFF = 1.02 × 78 × 12 × 0.155 = 148 kip

\[ M_{\text{max at the joint}} = -148 \times 12 \times \frac{1}{2} - \frac{1}{2} \times 0.43 \times 1 = -918.96 \text{ kip-ft} \]

3.7.2.2 Design Assumptions

Permanent erection bars were selected in this design example.

\[ f_{pu, \text{ for PT bars}} = 150 \text{ ksi} \]

\[ P_u \text{ of 1.375" dia. bar} = 1.58(150) = 237 \text{ kip} \]

\[ P_u \text{ of 1.25" dia. bar} = 1.25(150) = 187.5 \text{ kip} \]

\[ P_u \text{ of 1.0" dia. bar} = 0.85(150) = 127.5 \text{ kip} \]

Jacking force: 75% of GUTS

Check anchoring forces after anchor set for 1 ¼ in dia. PT bars.

Losses due to friction:

\[ \Delta F_{p,j} = F_{p,j} \left( 1 - e^{-(\kappa x + \mu \alpha)} \right) \quad \text{(LRFD5.9.5.2.2b-1)} \]

where

\[ F_{p,j} = \text{Force in the prestressing steel at jacking, (kip)} \]
\[ x = \text{Length of a prestressing tendon from the jacking end to any point under consideration, (ft)} \]
\[ \kappa = \text{Wobble coefficient, (ft}^{-1}) \]
\[ n = \text{Coefficient of friction (1/ rad)}; \]
\[ \alpha = \text{Sum of the absolute values of angular change of prestressing steel path from jacking end, (rad)} \]
\[ e = \text{Base of the Napierian logarithm} \]

Jacking force: \[ P_j = 0.75 \times 187.5 = 140.625 \text{ kip} \]

\[ L = 12 \text{ ft (segment length)} \]
\[ \kappa = 0.0002 \text{ per ft} \]
\[ \mu = 0.3 \]
\[ \alpha = 0.0 \]
Anchor set δ = 1/16 in 0.0052 ft

\[ \Delta P_f = 140.625 \times \left(1 - e^{-\left(0.0002 \times 12\right)}\right) \]

= 0.0337 kip

\[ \therefore P_f(\delta) = 140.625 - 0.0337 = 140.29 \text{ kip} \]

Friction loss is negligible.

Loss of stress due to anchor set \( E_s \varepsilon = 30,000 \left(\frac{0.0052}{12}\right) = 13 \text{ ksi} \)

\[ P_l = 140.624 - 1.25(13) = 124.375 \text{ kip (66% G.U.T.S)} \]

Therefore, anchoring forces, immediately after seating equal to 66% of GUTS
Try: 4 – 1 ¼” dia. top bars and
2 – 1 3/8” dia. bottom bars, as shown in Figure 3.79

\[ \therefore P_i \text{ top} = 4 \times 0.66 \times 187.5 = 495 \text{ kip} \]

\[ P_i \text{ bottom} = 2 \times 0.66 \times 237 = 312.84 \text{ kip} \]

\[ \sum P_i = 807.84 \text{ kip} \]

Compute CGS location relative to the top fiber

\[ 807.84 \times Y_s = 495 \times 0.5 + 312.84 \times (9 - 0.375) \]

\[ Y_s = 3.65 \text{ ft} \]
PT bars eccentricity = 3.65 – 3.4 = 0.25 ft (below C.G.C.)

a. Check joint stresses due to dead loads and PT bars

\[
|f_b| = -0.046 \text{ksi} > 0.03 \text{ksi} \quad \text{(LRFD 5.14.2.4.2)}
\]

\[
f_b = \sum P_i \frac{A_i}{S_b} - \sum P_e \frac{S_b}{S_b} - \frac{M_{DL}}{S_b}
\]

\[
= -11.478 \left( \frac{807.84 \times 0.25}{141.40} \right) - 918.96 \left( \frac{1}{141.40} \right)
\]

\[
= -11.478 - 1.428 - 6.450
\]

\[
= -19.406 \text{ksf} = -0.134 \text{ksi}
\]

\[
|f_b| = -0.134 \text{ksi} > 0.03 \text{ksi}
\]

\[
\text{Average stress} = \frac{0.046 + 0.134}{2} = 0.09 \text{ksi} > 0.04 \text{ksi} \quad \text{(LRFD 5.14.2.4.2)}
\]

b. Check stresses at the joint due to dead loads, PT bars and cantilever tendons

Tendon size: 4 – 12Ø0.6” strands.

\[
P_i = 0.7 \times 50.6 \times 48 = 1,968.96 \text{ kip}
\]

\[
\text{Tendon eccentricity} = 3.4 - 0.5 = 2.9 \text{ ft}
\]

Stress due to cantilever tendons:

\[
f_t = \sum \frac{P_i}{A_i} - \sum \frac{P_e}{S_t}
\]

\[
= \frac{1,968.96}{70.38} - \frac{1,968.96 \times 2.9}{232.89}
\]

\[
= -27.98 - 24.52
\]

\[
= -52.5 \text{ksf} = -0.3646 \text{ksi}
\]

\[
f_b = \sum \frac{P_i}{A_i} + \sum \frac{P_e}{S_b}
\]

\[
= -27.98 + \frac{1,968.96 \times 2.9}{232.89}
\]

\[
= -27.98 + 24.52
\]

\[
= -3.46 \text{ksf} = -0.024 \text{ksi}
\]

Tendon size: 2 – 12Ø0.6” strands. (50% less PT)

\[
f_t = 0.5(-0.3646) = -0.1823 \text{ksi}
\]

\[
f_b = 0.5(-0.024) = -0.012 \text{ksi}
\]
Summation of stresses.
For segments with 4 – 12Ø0.6” tendons

\[ \sum f_i = -0.046 - 0.3646 = -0.4106 \text{ ksi} \]
\[ |\sum f_i| = |-0.4106 \text{ ksi}| > -0.03 \text{ ksi} \]
\[ \sum f_i = -0.134 - 0.024 = -0.158 \text{ ksi} \]
\[ |\sum f_i| = |-0.158 \text{ ksi}| > 0.03 \text{ ksi} \]

For segments with 2 – 12Ø0.6” tendons

\[ \sum f_i = -0.046 - 0.1823 = -0.2283 \text{ ksi} \]
\[ |\sum f_i| = |-0.2283 \text{ ksi}| > -0.03 \text{ ksi} \]
\[ \sum f_i = -0.134 - 0.012 = -0.146 \text{ ksi} \]
\[ |\sum f_i| = |-0.146 \text{ ksi}| > 0.03 \text{ ksi} \]

Conclusion:
The proposed permanent PT bars satisfy the allowable joint stresses.

3.8 Detailing

3.8.1 Combined Transverse Bending and Longitudinal Shear
On the basis of previously determined shear reinforcement and flexural reinforcement, the standard practice has been to use the worst case of adding 50% of shear steel to 100% of the flexural steel, or 100% of the shear steel to 50% of the flexural steel.

A rational approach can also be used, where the compression strut in an equivalent truss model would be shifted to the extreme edge of the web. This compression would then be eccentric to a section through the web which would counteract an applied moment. If the applied moment were to exceed the amount that could be resisted in this manner, additional reinforcing could be added.

3.8.2 Shear Key Design
There are two types of shear keys in match-cast joints between precast segments:

- Web shear keys – Located on the faces of the webs of precast box girders. Corrugated multiple shear keys are preferred due to their superior performance. When designing shear keys, only web shear keys are considered in transferring the shear forces.
- Alignment keys – Located in the top and bottom flanges. Alignment keys are not expected to transfer the major shear forces; rather they facilitate the correct alignment of the two match-cast segments being erected in vertical and horizontal directions. For a single-cell box, normally a minimum of three alignment keys are required on the top slab and one on the bottom slab. However, alignment shear keys help in preventing local relative vertical displacement on the deck.
slab between two adjacent precast segments due to concentrated load on one side of the match cast joint. Therefore, in longer slabs spanning between two webs or longer cantilevers wings, it is necessary to provide more than one alignment shear key.

Both shear and alignment keys should not be located in the tendon duct zones. (see Figure 3.83)

The design of web shear keys should satisfy two design criteria:

1. Geometric design: As per LRFD Figure 5.14.2.4.2-1, the total depth of shear keys extends approximately 75% of the section depth and at least 75% of the web thickness.

2. Shear strength design: As per AASHTO Standards Specifications, 17th Edition (AASHTO 2002), Article 9.20.1.5, reverse shearing stresses should be considered in the shear key design. At the time of erection, shear stress carried by the shear key should not exceed $2\sqrt{f'_c}$ (psi). Alternatively, strength of the shear key could also be computed in accordance with article 12.2.21 of AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges, Second Edition (AASHTO 1999). However, the AASHTO Guide Specification shear key provision was developed for dry joints only. Note that dry joint is no longer permitted by AASHTO LRFD Specifications.

Shear key design example was illustrated in Figure 3.80

1. Geometric consideration (see Figures 3.81 to 3.83)
   \[ h = 9 \text{ ft} \]
   \[ \text{Shear key depth} = 0.75 \times 9 \text{ ft} = 6.75 \text{ ft} \]
   \[ b_w = 16 \text{ in} \]
   \[ \text{Shear key width} = 0.75 \times 16 = 12 \text{ in} \]

2. Shear strength design of the shear keys
   AASHTO LRFD Bridge Design Specification does not specify any guideline on the strength design of shear keys. Use AASHTO Standard Specifications, article 9.20.1.5.

   a. AASHTO Standard Specifications, article 9.20.1.5
      \[ \begin{align*}
         V_u &= 1.1(V_{DC} + \text{DIFF})
      \end{align*} \]

   \[ \text{FIGURE 3.80} \quad \text{Precast segment being erected.} \]

   \[ \text{FIGURE 3.81} \quad \text{Details of shear keys.} \]
where 
\[ V_{DC} = \text{shear force due to self-weight of one typical segment (kips)} \]
\[ = 78 \times 12 \times 0.155 = 145 \text{ kip} \]

DIFF = 2% of \( V_{DC} \)

\[ V_u = 1.1 \times 145 \times 1.02 = 162.8 \text{ kip} \]

\[ V_n = V_c \]

\[ V_u / \phi = V_c \]

Consider one web only,

\[ V_t = 0.5 V_u / \phi \text{, per web,} \]

\[ V_t = A_k \cdot v \text{, per key,} \]

Notes:
1. \( 1 1/4'' \leq a \geq \) twice the diameter of the top size aggregate.
2. As per AASHTO LRFD specifications Fig. 5.14.2.4.2-1.

**FIGURE 3.82** Web shear key detail.
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where

\[ \phi = 0.9 \text{ article 9.14 of AASHTO Standard Specifications} \]

\[ \nu = \text{allowable shear stress} \]

\[ \nu = 2 \sqrt{f'_c} \text{ (psi)} \]

\[ A_k = \text{shear area of one key} \]

\[ A_k = 3.5 \times (12) = 42 \text{ in}^2 \]

\[ V_{c \text{ per web}} = (0.5 \times 162.8)/0.9 = 90.44 \text{ kip} \]

\[ V_{c \text{ per key}} = 42 \times 2 \sqrt{6000} = 6506.6 \text{ lbs} = 6.5 \text{ kip} \]

Number of male keys required per web = \( \frac{90.44}{6.5} = 13.9 \text{ say 14 keys} \)

3.8.3 Strut and Tie Model

In segmental bridge design and detailing, strut and tie model is extensively used in studying the flow of forces from one structural element to the other. The flow of forces from the box girder to the diaphragm, bearings, pier cap, pier column, pile cap to pile foundations can be modeled by the strut and tie model. The strut and tie model is a truss system analogy applied to concrete members consisting of compression and tension members, and is tied together by nodes. Some examples of strut and tie models are shown in Figures 3.84 through 3.91.
FIGURE 3.84 Transfer of moment from the box girder to the pier column (From Menn, C. 1986. *Prestressed Concrete Bridges*; Birkhauser-Verlag, Boston, MA, 1986.)

\[ \Delta T = \Delta C \]
\[ \Delta M = \Delta T \times h_0 \]

FIGURE 3.85 Transfer of forces from the diaphragm to a single bearing. (From Menn, C. 1986. *Prestressed Concrete Bridges*; Birkhauser-Verlag, Boston, MA, 1986.)
3.9 Durability

3.9.1 Durability Problems of Posttensioned Bridges in the United States

After the findings of corrosion in posttensioning tendons in some of Florida's bridges in 1999 to 2000, durability of posttensioned concrete bridges in the United States has become a great concern to owners and bridge engineers around the country. Other deficiencies of posttensioned bridges such as cracked polyethylene ducts and grout voids were also found in other states. For half a century, since the construction of Walnut Lane Bridge in Philadelphia (1949–1950), the first posttensioned bridge in the United States, this type of bridge has enjoyed its popularity as an economical and durable structural system that requires minimum maintenance.

In the summer of 1999, one of the external tendons in the superstructure box girder of the Niles Channel Bridge in Florida Keys was found to have failed due to corrosion. The bridge was constructed in early 1983 and is believed to be one of the first span-by-span segmentally erected concrete bridges in Florida (Fib 2001).

In August 2000, during a routine inspection of the Mid-Bay Bridge located in Destin, Florida, a posttensioning tendon in span 28 was found in a significant state of distress. The polyethylene external duct was cracked and several strands were fractured. Further inspection of the bridge revealed that a posttensioned tendon in Span 57 had failed completely at the north end of the tendon. The tendon pulled out from the expansion joint diaphragm as a result of severe corrosion of the tendon in the anchorage.
area. After extensive investigation and inspection, it was found that 11 tendons required replacement. The bridge was constructed in 1992 using span-by-span precast segmental construction. In addition to tendon corrosion, cracked polyethylene ducts and grout voids were also discovered (FDOT 2001).

In September 2000, during a special inspection of the high level approach columns in the Sunshine Skyway Bridge in St. Petersburg, Florida, it was discovered that severe corrosion resulted in the failure of 11 strands of the southeast external vertical tendon located in column 133 northbound (PBQD 2002). This finding triggered an extensive investigation of all other high-level approach columns, the bridge superstructure, and cable anchorage of the main span bridge. The investigation of the rest of the columns revealed severe tendon corrosion in the anchorages and at the base of the columns, including cracked polyethylene duct, grout void, and grout chloride contamination. The 76 high-level approach columns have been repaired, including deficiencies found in the superstructure external tendons. The bridge was constructed in 1982 to 1987 and utilized the precast segmental construction method, including for the high-level approach columns.

FIGURE 3.87 Transfer of torsion moment from the box girder through the diaphragm to bearings. (From Menn, C. 1986. Prestressed Concrete Bridges; Birkhauser-Verlag, Boston, MA, 1986.)
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Shear reinforcement

Additional web reinforcement

\[ A_s \cdot f_{py} \geq 0.2 \cdot A_p \cdot f_{py} \]

FIGURE 3.88 Transfer of blister forces to the web. (From Menn, C. 1986. Prestressed Concrete Bridges; Birkhauser-Verlag, Boston, MA, 1986.)

Span tendons

Stress diaphragm in the section

Thrusted tendon (radial forces)

FIGURE 3.89 Radial forces in curved bottom slab. (From Mathivat, J. The Cantilever Construction of Prestressed Concrete Bridges; John Wiley & Sons, New York, NY, 1979.)
The findings of the different bridges in Florida raised concerns about the grouting situation in the Central Artery/Tunnel project in Boston where there are considerable numbers of segmental and cast-in-place posttensioned concrete structures. It was important to determine if the Central Artery structures have grout voids and whether or not the tendons are corroded, although the structures are relatively young. Posttensioned tendons inspection was conducted at 380 locations. This is less than 1.5% of...
the total number of tendons on the project. The initial investigation revealed excessive amounts of grout voids but no corrosion of strands.

### 3.9.2 Durability Problems of Posttensioned Bridges around the World

Signs of durability problems in Europe were discovered about 20 to 30 years prior to the discovery of posttensioned corrosion problems in the United States.

In December 1985, a single-span precast segmental bridge in the United Kingdom, namely Ynys-y-gwas, collapsed without warning (The Highway Agency, Setra, TRL, and LCPC. 1999). Since then a sample of nine segmental bridges were inspected; grout voids in seven bridges and severely corroded tendons in two bridges were discovered. In the late 1980s and early 1990s, about a dozen posttensioned concrete bridges in the United Kingdom were discovered with serious tendon corrosion, which required major repairs or replacement. The inspection results led to the UK Department of Transport (currently called Highway Agency) initiated a ban on internally grouted posttensioned concrete bridges in September 1992. The moratorium was lifted for cast-in-place posttensioned concrete bridges in 1996. However, the moratorium for precast segmental posttensioned concrete bridges, including segmental bridges with epoxy joints, is still in place. In 1992 the UK Department of Transport launched a five-year special inspection program for all existing posttensioned concrete bridges located on the Trunk Road. As a result of the inspection program, 447 posttensioned concrete bridges were completely inspected and documented.

In 1970, the first serious sign of durability problems of posttensioned concrete bridges in France was discovered with the finding of several concrete cracks in the side span of the Chazy Bridge caused by severe tendon corrosion. The bridge was demolished and reconstructed in 1972. Additionally, tendon corrosion, grout deficiency, and other posttensioning system defects were also found in the Choisy-le-Roi Bridge, the Vaux Sur Seine Bridge, the Port a Binson Bridge, the Villeneuve Saint-Georges Bridge, the Can Bia Bridge, the Saint-Cloud Viaduct, the bridge over the Durance, and the Riviere d’Abord Bridge, including the first generation of simple-span posttensioned bridges constructed in the period between 1946 and 1960 (Fib 2001).

Japan Highway Public Corporation conducted inspections and investigated 120 posttensioned concrete bridges. The results showed 31% of the tendons investigated have grout deficiency such as no grout, imperfect grout, and grout voids. Other deficiencies such as tendon corrosion, reinforcement corrosion, concrete cracks, and spalling were also found. As a result of these findings, the Japan Highway Public Corporation has placed a moratorium on new construction of grouted posttensioned bridges (Fib 2001).

Other countries such as Germany, Austria, and Italy also have their share of corrosion problems with their posttensioned bridges. For instance, the collapse of Congress Hall in Berlin is one of the most spectacular posttensioned structure failures in Germany, although it was not a posttensioned bridge. FIB Task Group 9.5 reported that several posttensioned concrete bridges in Germany were affected by severe tendon corrosion, for example failure of the flyover at the Heerdter in Dusseldorf in 1976, tendon corrosion of the flyover junction in Berlin-Schmargendorf, tendon corrosion of the A73 motorway at South Nurenberg, and tendon corrosion of the bridge over Muckbachtal on motorway Wurzburg-Heilbronn. Consequently, German Federal Ministry for Transport and Construction has placed a moratorium on internal tendons in the webs of posttensioned bridges, with the exception of replaceable external tendons and internal tendons in the top and bottom flanges of box girder.

### 3.9.3 Lessons Learned

The United Kingdom is one of the few countries in the world that has undertaken an extensive study and investigation of its posttensioned concrete bridges. As mentioned above, from the Special Inspection Program, 447 posttensioned concrete bridges were systematically documented. Such a study allows the
UK Highway Agency to determine the important factors affecting their posttensioned concrete bridges. Although there are common problems associated with durability of posttensioned bridges shared between countries, it is believed that each country has its own unique problems. The study concluded that corrosion of posttensioned tendons in internally grouted duct has occurred in a small number of posttensioned concrete highway bridges in the United Kingdom. Therefore, the report finds that the majority of the structures have a good record of durability.

In the United States, special inspection and investigation of posttensioned bridges were conducted in the States of Florida, Texas, Virginia, Georgia, Mississippi, Delaware, Kansas, South Carolina, Indiana, Iowa, Rhode Island, and Massachusetts. While there is no national guideline for this type of investigation, the Florida Department of Transportation (FDOT) has become the leader for this type of investigation in the United States. Unfortunately, there is no single coordinated effort in the United States to collect and study the inspections/investigations completed from different States. To date, the findings of posttensioned concrete bridges investigations in the United States, such as Florida, Texas and other states are very similar to the findings in the European countries’ investigations.

The following is the brief summary findings of posttensioned bridges investigation in the United States:

- Cracked polyethylene duct of external tendon
- Grout voids in the posttensioned duct and anchorages as result of water bleed
- Grout voids as a result of poor construction practice, quality control, design, and detailing
- Tendon corrosion and failure as a result of water and oxygen intrusion due to failure of tendon protection system
- Tendon corrosion as a result of unsuitable grout materials and chloride contamination
- Tendon corrosion as a result of shortcomings in specifications and corrosion detection methods

Grout voids do not necessarily compromise the durability of posttensioned structures, as long as one or more of the tendon protection systems are still intact and undamaged. This has been verified in the investigation findings of posttensioned bridges in the United Kingdom and in the United States.

It is believed that there are three factors that may have contributed to the durability problems of posttensioned bridges today:

1. The original philosophy of prestressed concrete design of full prestressing (no tension allowed at working loads) created a perception that prestressed concrete should be crack free and therefore required minimal or no maintenance.
2. Lack of historical data and testing on emerging new technologies in posttensioned bridge designs and construction methods.
3. Generally, grouted tendons are perceived as adequate corrosion protection to the prestressing steel, notwithstanding the inability to inspect the condition and quality of the grout inside the ducts.

### 3.9.4 New Direction for the Next Generation of Posttensioned Concrete Bridges

While the well-known 1992 UK DOT moratorium on posttensioned bridges was considered an overreaction by most countries at that time, the moratorium has impacted the concrete bridge industries of the world in very positive ways. It has also challenged the Bridge Engineering Society and Industry to review and make revisions to the construction and materials specifications, including design and detailing of posttensioned bridges. Technical Report 47 (TR 47) is one of the most important documents ever produced by the UK Concrete Society to deal with durability of posttensioned concrete bridges in response to the moratorium (The Concrete Society 1996).
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FDOT and TxDOT (Texas Department of Transportation) are leading in developing and implementing the new direction for the future generation of posttensioned concrete bridges in the United States. Similar to the United Kingdom, FDOT has rewritten its posttensioned grouting specifications, posttensioned system specifications, quality control manuals, and certification requirements, including semistandard posttensioning details. The Posttensioning Institute (PTI) also has rewritten its grouting specifications. In support of FDOT and PTI, the American Segmental Bridge Institute (ASBI) contributed in improving grouting practice, workmanship, quality control, and posttensioning details by setting up ASBI Grouting Training Certification Seminars conducted once a year since 2001. The new posttensioned concrete bridge projects in Florida have fully implemented the new posttensioned specifications and details since 2003 (FDOT 2003).

3.9.5 Conclusions

The durability problems found in Florida and in other parts of the country do not necessarily represent the condition of all posttensioned concrete bridges in the United States. Observe that almost all of the posttensioned bridges mentioned above that are affected by corrosion are located in the very corrosive environment of Florida coastal areas and are associated mostly with the precast segmental construction method. A durable posttensioned structure can be constructed to last the service life provided the required improvements are done across the board including materials, construction methods, design, detailing, workmanship, quality assurance, quality control, corrosion detection system, inspection, and preventive maintenance. Neglecting any one of the above will indeed compromise the durability of posttensioned concrete bridges. Similar to other types of bridges, posttensioned concrete bridges require routine inspection and maintenance.

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