CHAPTER 5.3 PRECAST PRETENSIONED CONCRETE I-GIRDERS

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5.3.1 INTRODUCTION

Precast concrete elements such as girders, piles, deck panels, and pavement are being used with increasing frequency in California. This chapter focuses exclusively on precast pretensioned concrete I-shape girders, referred to herein as PC girders. PC box girders and PC voided slabs are discussed in Chapters 5.4 and 5.5, respectively.

PC girders facilitate rapid construction of a bridge because they are fabricated off-site and then transported and erected into place at the job site. Once the deck is poured and cured, the structural section becomes composite, minimizing deflections. Because PC girders require little to no falsework, they are a preferred solution for jobs where Accelerated Bridge Construction (ABC) is sought, where speed of construction, minimal traffic disruption, and/or reduced environmental impact is required, and where temporary construction clearance is limited. PC girders employ high performance concrete for strength, durability, and/or constructability and tend to be more economical and competitive when significant repeatability exists on a job (i.e., economy of scale). The use of PC girders in the California Highway Bridge System has increased rapidly in recent years (Figure 5.3.1-1).

A) Pretensioned bulb-tee girders B) Pretensioned wide-flange girder

Figure 5.3.1-1 Example of Precast Pretensioned Concrete I-Girder Sections

Similar to cast-in-place (CIP) post-tensioned (PT) box girders, PC girders are prestressed to produce a tailored stress distribution along the member at service level to help prevent flexural cracking. For member efficiency, the girders have pre-compressed tensile zones-regions such as the bottom face of the girder at midspan where compression is induced to counteract tension due to expected gravity loads (e.g., self-weight, superimposed dead loads such as deck weight, barrier weight, and overlay, as well as live loads). To achieve this, PC girders employ prestressing strands that are stressed before the concrete is placed in the forms and hardens. This is in contrast to PT girders, in which the tendons are stressed after the concrete hardens. However, PC girders may also be pretensioned, then post-tensioned, and are sometimes spliced together to form a single span or continuous superstructure.

As shown in Figure 5.3.1-2, pretensioning requires the use of a stressing bed, often several hundred feet long for efficient casting of a series of members in a long line, and using abutments, stressing stands, jacks, and hold-downs/hold-ups to produce the desired prestressing profile. The transfer of strand force to the concrete members by bond is typically evident by the upward deflection (camber) of members when the strands are detensioned (cut or burned) at the member ends. Steam curing of members allows for a rapid turnover of forms (typically one-day cycle or less) and cost efficiency. Control during fabrication of PC girders also permits the use of quality materials and provides many benefits such as higher strength materials (e.g., f'_{ci}, f'_c) and modulus of elasticity, as well as reduced creep, shrinkage, and permeability. Article 5.5.4.2 of CA Amendments to AASHTO LRFD Bridge Design Specifications (Caltrans, 2019a) takes advantage of this higher quality control and workmanship, thus increases the resistance factor, ϕ, for tensioned-controlled sections from 0.95 for CIP PT members to 1.0 for PC girders.

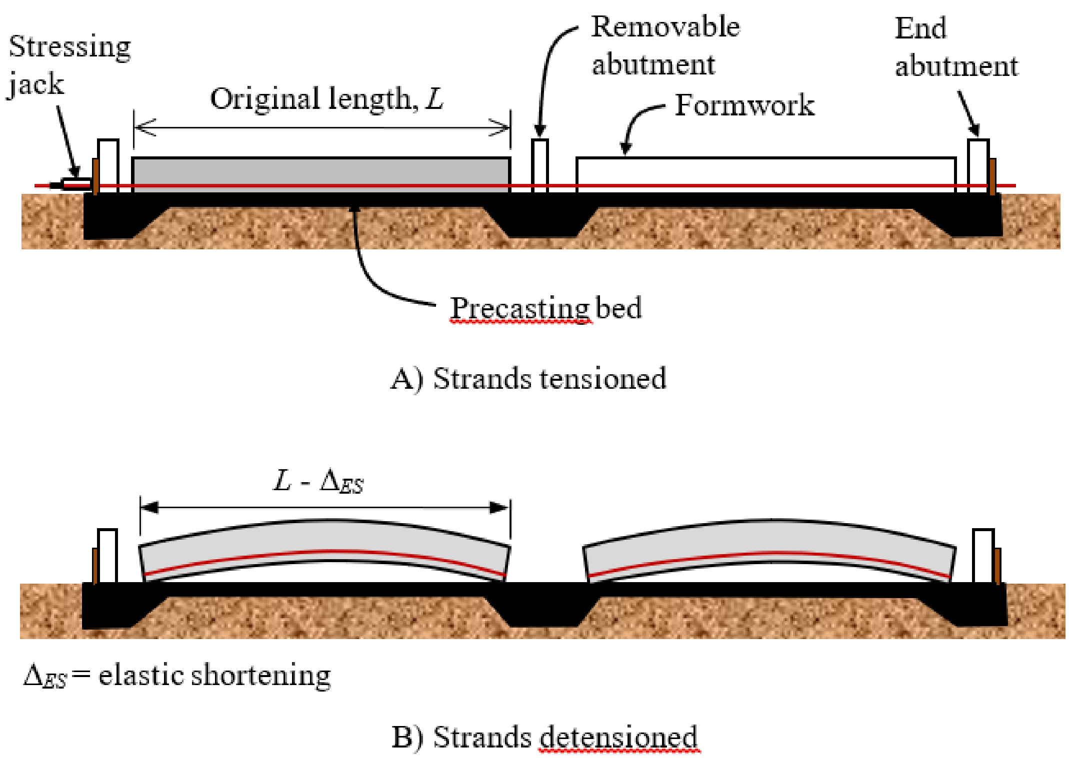


Figure 5.3.1-2 Pretensioning of Members with Straight Strands on Stressing Bed

5.3.2 PRECAST GIRDER FEATURES

5.3.2.1 Typical Sections and Span Ranges

The designer may select from a wide variety of standard sections, as described in BDM 5.3. Girder sections not covered in this section are considered non-standard and must be approved by the Type Selection Meeting.

Figure 5.3.2-1 shows representative PC girder sections, and Table 5.3.2-1 lists possible and preferred span lengths for eight common PC girder types, including four standard California girders (I, bulb-tee, bath-tub, and wide-flange) and the precast voided slab, as well as three other PC girders (box, delta, and double-tee).

Table 5.3.2-1 PC Girder Types and Span Lengths (BDM 5.3)

Girder Type	Possible Span Length(ft)	Preferred Span Length(ft)
California I-girder	50 to 125	50 to 95
California bulb-tee girder	80 to 150	95 to 150
California bath-tub girder	80 to 150	80 to 120
California wide-flange girder	80 to 200	80 to 180
Precast voided slab	20 to 70	20 to 50
Precast box girder	40 to 120	40 to 100
Precast delta girder	60 to 120	60 to 100
Precast double-tee girder	30 to 100	30 to 60

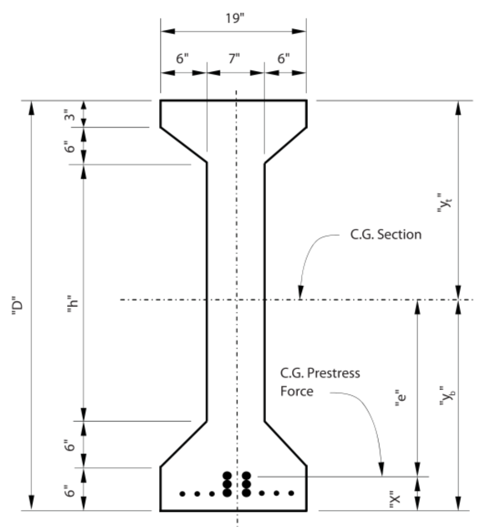
Among these girders, the I-girder is commonly used and has been in use in California for nearly 60 years. With bridge span lengths ranging from 50 ft to 125 ft, the I-girder typically uses a depth-to-span ratio of approximately 0.05 to 0.055 for simple spans and approximately 0.045 to 0.05 for multi-span structures made continuous for live load.

The bulb-tee and bath-tub (or U-shape) girders are targeted for bridge spans up to 150 ft. The depth-to-span ratio is slightly smaller than that for I-girders: 0.045 to 0.05 for simple spans and 0.04 to 0.045 for continuous structures, respectively. However, due to the weight limits for economical hauling, the length of bath-tub girders is usually restricted to a range of 100 ft to 120 ft.

The California wide-flange girder (Figure 5.3.2-2) was recently developed in coordination with California precasters to produce more efficient bottom and top flange areas that permit design for spans up to 200 ft, with a depth-span ratio of 0.045 (simple) and 0.04 (continuous). The larger bottom bulb accommodates nearly 20% more strands than the standard California bulb-tee and, due to its shape, provides enhanced handling and erection stability for longer spans. Greater economy is also anticipated due to larger girder spacing and reduction in girder lines. Standard sections have been developed for both pretensioning alone, as well as combined pre- and post-tensioned sections. For longer span lengths such as 125’ or more, special permits for hauling, trucking routes, and erection must be verified.

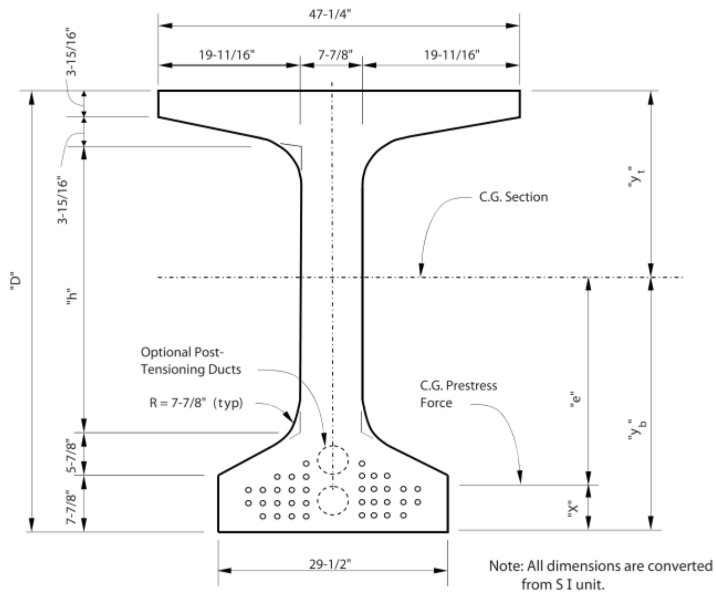
Other girders that are less commonly used include girders with trapezoidal, double-tee, and rectangular cross sections as well as box girders. These are sometimes used for cost effectiveness and aesthetics. Precast box girders are often used for railway systems and relatively short span lengths ranging from 40 ft to 100 ft.

It should be noted that using the given bridge depth-to-span ratios to determine the girder section is approximate but is usually a reasonable starting point for initial design and cost estimates. Normally, girder spacing is set at approximately 1.25 to 1.75 times the bridge superstructure depth. When a shallow girder depth is required, girder spacing may have to be reduced to satisfy all design criteria, which may result in increased cost.



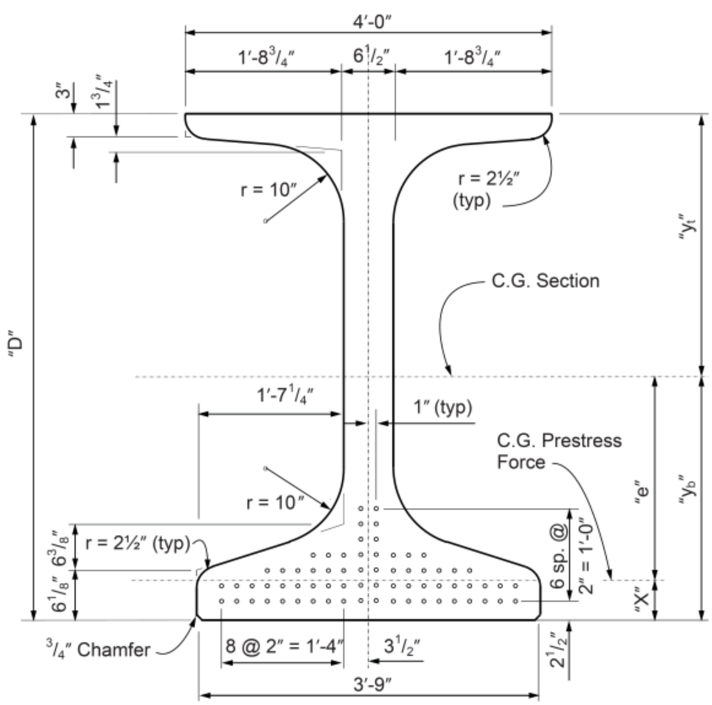
A) I-girder

Figure 5.3.2-1 Example PC Girder Sections (BDM 5.3)



B) Bulb-tee

Figure 5.3.2-1 Example PC Girder Sections (BDM 5.3)



C) Wide-Flange

Figure 5.3.2-1 Example PC Girder Sections (BDM 5.3)

Figure 5.3.2-2 California Wide-Flange Girders

5.3.2.2 Primary Characteristics of Precast Girder Design

The heart of the prestressed concrete design philosophy is the positioning of the prestressing strands within the PC girder: the center of gravity of the strands (*CGS*) is deliberately offset from the center of gravity of the concrete section (*CGC*) to establish an eccentricity, defined as the distance between the *CGS* and *CGC* at a section. This eccentricity produces a beneficial tailored flexural stress distribution along the length of the member to counteract the flexural tension expected from gravity loads. The largest eccentricity is provided at locations where tension is expected to be the greatest (e.g., at midspan of simple span girder).

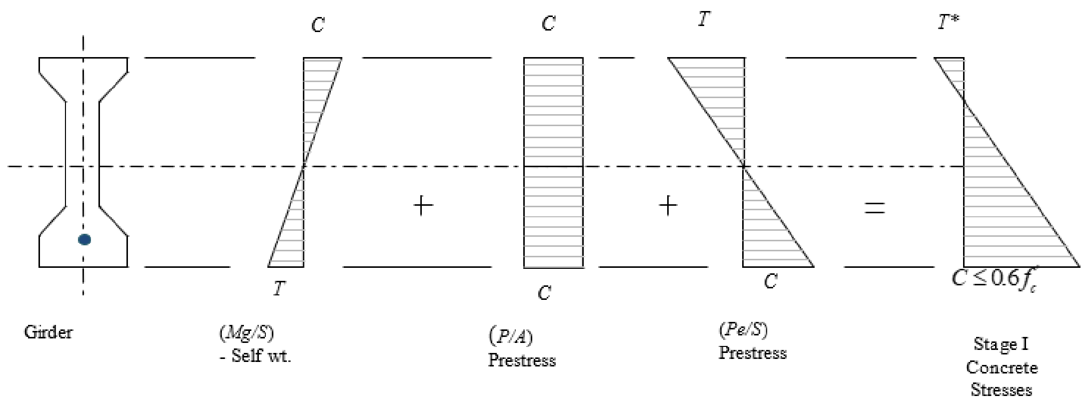
For PC girder design, the following three basic stages are addressed: Transfer, Service, and Ultimate.

- Transfer refers to the stage at which the tensile force in the strands is transferred to the PC girder, by cutting or detensioning the strands after a minimum girder concrete strength has been achieved. Because the girder is simply supported and only self-weight acts with the prestressing at this stage, the most critical stresses typically occur at the ends of the girder or harping points (also known as drape points). Both tensile and compressive stresses should be checked at these locations against AASHTO LRFD Bridge Design Specifications, 8th Edition (AASHTO, 2017) and California Amendments to AASHTO LRFD Bridge Design Specifications, 8th Edition (Caltrans, 2019a) stress limits.
- Service refers to the stage at which girder and deck self-weight act on the non-composite girder, together with additional dead loads (e.g., barrier and wearing surface) and live load on the composite section. This stage is checked using the AASHTO-CA BDS-8 Service I and III load combinations (AASHTO, 2017). Per Caltrans Amendments Table 5.9.2.3.2b-1 (Caltrans, 2019a), the girder must also be designed to prevent tension in the precompressed tensile zones ("zero tension") due to permanent loads.

- Ultimate refers to the Strength Limit State. Flexural and shear strengths are provided to meet all factored moments and forces, including the Caltrans P-15 design truck (Strength II load combination).

In service limit state design, the concrete stresses change at various loading stages. In general, there are three major stages that need to be considered in the design, and these stages are described as follows:

- Stage I: Cast and stress girder (transfer) (Figure 5.3.2-3):
  - Strands are stressed to jacking force within form. Girder concrete is cast. Once concrete gains sufficient strength, strands are cut, transferring prestressing force to the girder.
  - Girder self-weight is supported by the PC girder alone.
  - This transfer stage is a temporary condition. For normal weight concrete, tensile stresses are limited to $0.0948\sqrt{f'_{ci}} \leq 0.2$ ksi for section without bonded reinforcement or $0.24\sqrt{f'_{ci}}$ for section with reinforcement sufficient to resist the tensile force in the concrete per Table 5.9.2.3.1b-1 (AASHTO, 2017). The compressive stresses are governed by limits in Article 5.9.2.3.1a (AASHTO, 2017).



* $T \leq 0.0948\sqrt{f'_{ci}}$ or 0.2 ksi for section without bonded reinforcement

* $T \leq 0.24\sqrt{f'_{ci}}$ for section with reinforcement sufficient to resist concrete tensile force

Figure 5.3.2-3 Representative Concrete Flexural Stress Distribution at Stage I (Transfer)

- **Stage IIA: Erect girder and cast deck slab (Figure 5.3.2-4):**
  - Girders are transported to project site and erected on structure supports. Diaphragms and concrete deck are cast.
  - When deck concrete is wet, deck slab does not contribute to section modulus for flexural resistance.
  - Temporary construction loads for machinery (e.g., Bidwell) need to be included.

- Girder self-weight plus weight of diaphragms and deck are supported by the PC girder alone.
- This stage is a temporary condition. Tensile and compressive stresses are governed by the limits in Article 5.9.2.3.1.

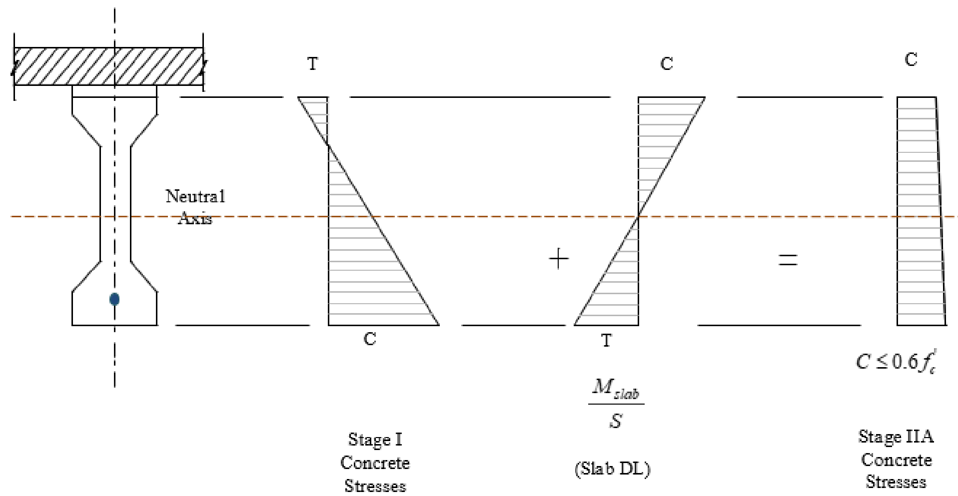


Figure 5.3.2-4 Representative Concrete Flexural Stress Distribution at Stage IIA (Erection and Deck Pour)

- **Stage IIB: Construct barrier rails (Figure 5.3.2-5)**
  - Deck concrete hardens and barrier rails are constructed. The girder and deck act together as a composite section.
  - Girder self-weight plus weight of diaphragms and deck are supported by the PC girder alone and additional dead load (haunch and barrier rails) is supported by the composite section.
  - Tensile and compressive stresses are governed by the limits in Article 5.9.2.3.1 (AASHTO, 2017).

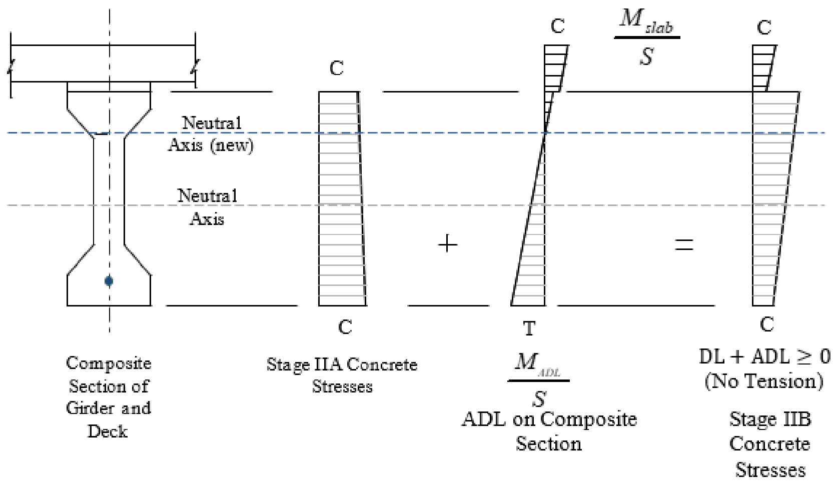


Figure 5.3.2-5 Representative Concrete Flexural Stress Distribution at Stage IIB (Barrier Rail Construction)

- **Stage III: Open to traffic (Figure 5.3.2-6):**
  - Girder and deck continue to act as a composite section.
  - Girder self-weight plus weight of diaphragms and deck are supported by the PC girder alone. Additional dead load (haunch and barrier rails) and live loads are supported by the composite section.
  - This stage is a permanent condition. Compressive and tensile stresses are governed by the limits in LRFD Specifications Table 5.9.2.3.2a-1 (AASHTO, 2017) and Caltrans Amendments Table 5.9.2.3.2b-1 (Caltrans, 2019a), respectively.

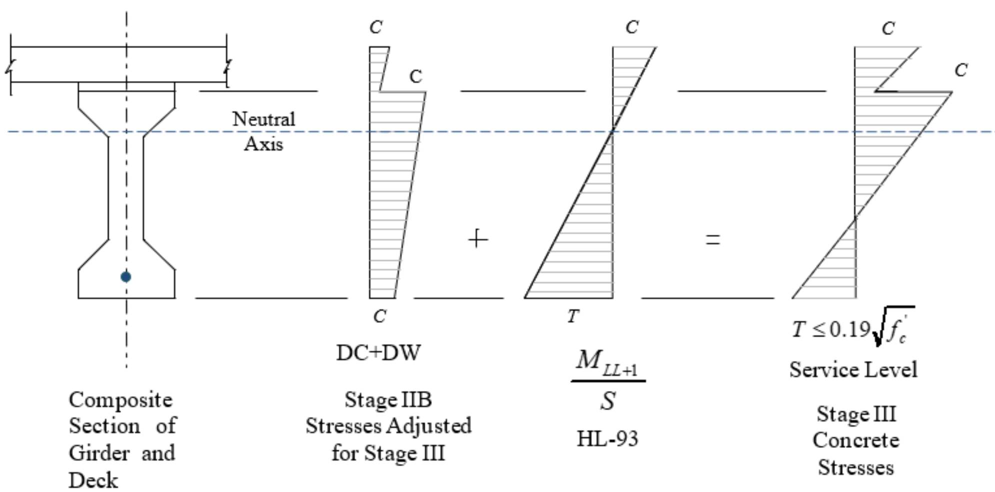


Figure 5.3.2-6 Representative Concrete Flexural Stress Distribution at Stage III (Open to Traffic)

5.3.2.3 Methods to Vary Strand Eccentricity and Force

Efficient design of PC girders typically requires varying the strand eccentricity along the length of the member and/or limiting the strand force at transfer. PC girders are fabricated, transported, and initially installed as simply supported segments. For a simply supported girder with straight strands, the large eccentricity between the *CGS* and the *CGC* section helps reduce tension and possible cracking at midspan at service level. However, excessive flexural tensile stresses may develop at the top of the girder segments near the ends, where counteracting flexural stresses due to self-weight are minimal. Excessive flexural compressive stresses may similarly develop. The critical location near the ends is at the transfer length, the distance from the end of the girder at which the strand force is fully developed. For this temporary condition, Table 5.9.2.3.1b-1 of LRFD Specifications (AASHTO, 2017) specifies appropriate stress limits to mitigate cracking and compression failure.

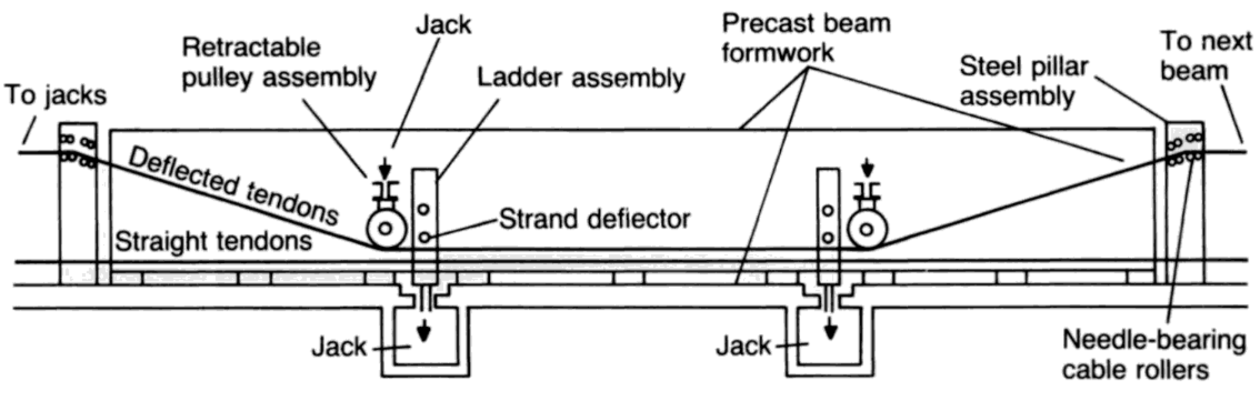


Figure 5.3.2-7 Drapped Strand Profile (Pritchard, 1992)

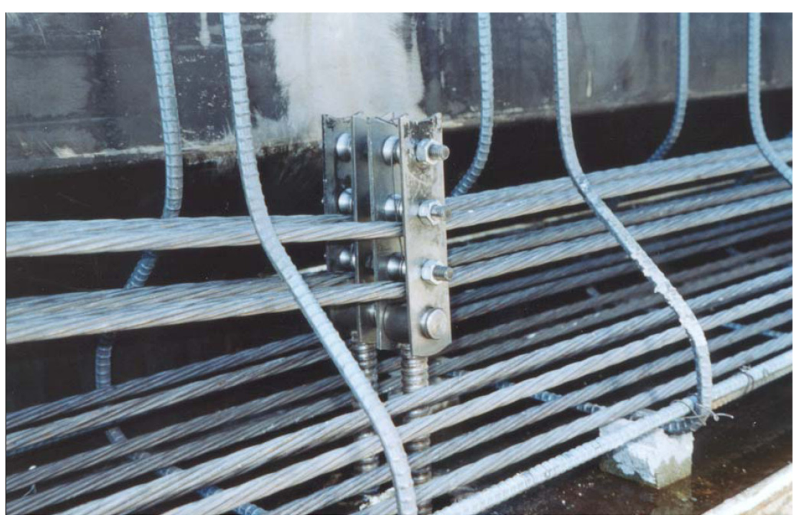


Figure 5.3.2-8 Hold-Down Assembly in Stressing Bed (Ma and Schendel, 2009)

To reduce the tensile and compressive stresses at the ends of girders, the designer normally considers two primary methods, both of which are used in California:

- Harping (or draping) strands to reduce the strand eccentricity (Figures 5.3.2-7 and 5.3.2-8):
  - Advantages of harping include:
    - Flexural design efficiencies due to the strand *CGS* achieving a profile corresponding to the moment envelope
    - Reduction of eccentricity at member ends to control concrete stresses at these critical regions at transfer
    - Additional shear capacity contributed by the vertical component of the prestress force in the harped strands
  - Disadvantages of harping include:
    - Safety issues and precaster ability to economically deflect and anchor harped strands
    - Slightly higher cost for fabrication and embedded hold-down devices
    - Beam form patching to accommodate variable hold down locations
- Debonding (or shielding) select strands at the member ends to reduce the transfer prestress force (Figure 5.3.2-9):
  - Advantages of debonding include:
    - Reduction in concrete stresses at member ends
    - Simpler fabrication by the use of only straight strands in the stressing bed
    - Elimination of hold-down devices
  - Disadvantages of debonding include:
    - Potential increase in design compressive strength of concrete
    - Increased design effort to determine debonding patterns, shear reinforcement, and camber

A) Single strand sheathing

B) Debonded strands in PC girder

Figure 5.3.2-9 Plastic Sheathing Used for Debonding Strand

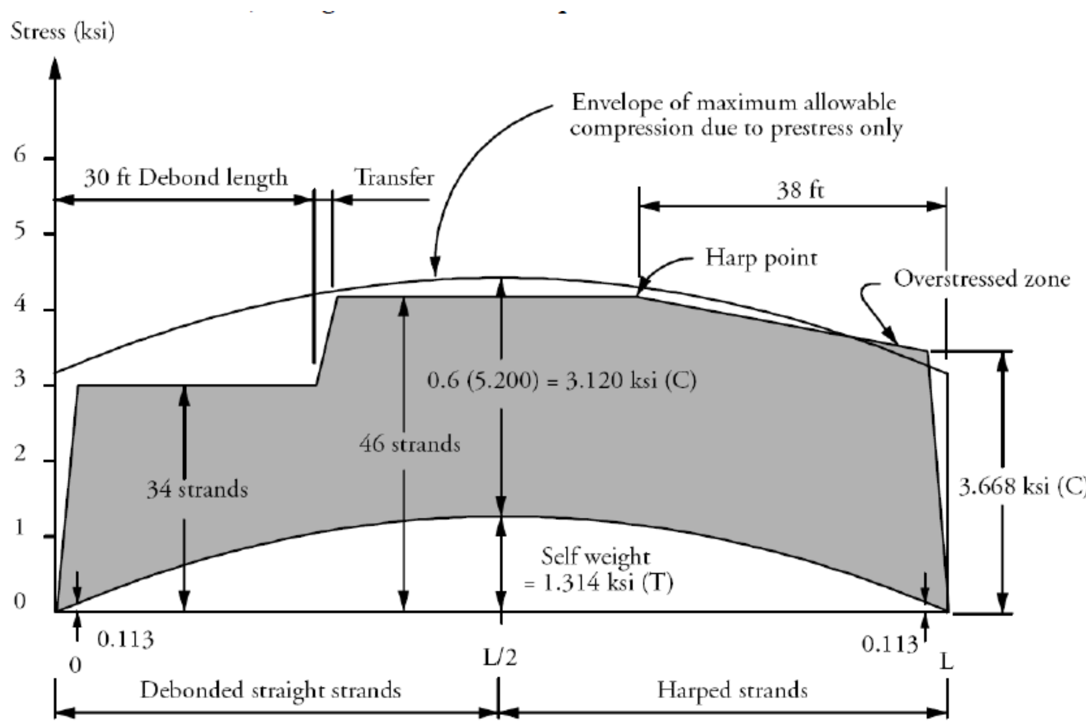


Figure 5.3.2-10 Example of Bottom Fiber Stress Distribution at Transfer: Harping vs. Debonding (PCI Bridge Design Manual 2014)

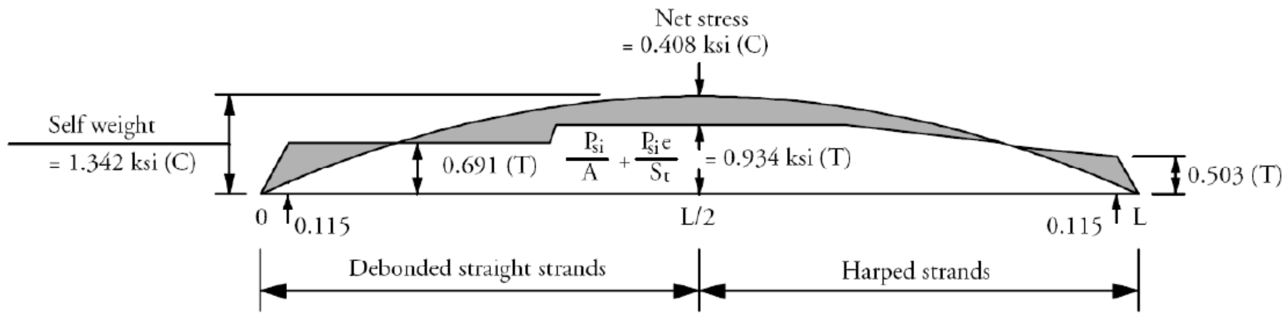


Figure 5.3.2-11 Example of Top Fiber Stress Distribution at Transfer: Harping vs. Debonding (PCI Bridge Design Manual 2014)

By draping the strands in a PC girder, the eccentricity can be varied in linear segments along the length of the girder by mechanically deflecting some of the stressed strands in the casting beds prior to casting using hold-downs and hold-ups, as shown in Figures 5.3.2-7 and 5.3.2-8. Although draping is limited to strands within the web, only a portion of the strands typically needs to be draped to achieve the required eccentricity at girder ends. Typically, the drape points are located between approximately 0.33*L* and 0.4*L*. Some fabricators may not have suitable equipment for all drape profiles. In addition, the drape angle must be limited to ensure that jacking requirements and hold-down forces do not exceed available capacity. The example patterns in Figures 5.3.2-10. and 5.3.2-11 provide a comparison of the bottom and top fiber stresses associated with harped and debonded strands.

Alternatively, the designer may choose to limit transfer stresses by reducing the prestress force through debonding strands along a portion of the girder length at member ends. This is known as partial debonding. Figure 5.3.2-9 shows debonding of a strand by encasing the strand in a plastic sheathing. Debonding strand prevents the prestressing force from developing in the debonded region and causes the critical section for stresses to shift a transfer length (i.e., 60 strand diameters, per LRFD Specifications) beyond the end of debonding. Caltrans Amendments (Caltrans, 2019a) limit the number of partially debonded strands to 33% of the total number of strands and the number of debonded strands in any horizontal row to 50% of the strands in that row. Increases in development length at ultimate are also addressed in Article 5.9.4.3.3 (AASHTO, 2017).

Due to the limitations in number of debonded strands at the girder bottom, the temporary stress at girder top at the ends may still exceed the allowable stress limits, especially for longer span girders. One solution is to use temporary strands at the girder tops that are shielded along the member length except at the girder ends. These strands can be cut at a later stage such as erection, when they are no longer needed, by providing an access pocket formed in the girder top.

5.3.3 PRECAST BRIDGE TYPES

There are three main PC bridge types: i) precast pretensioned girders, ii) precast post-tensioned spliced girders, and iii) precast segmental girders. Table 5.3.3-1 summarizes the typical span lengths for these bridge types.

Table 5.3.3-1 Precast Bridge Types and Span Lengths (BDM 5.3)

Bridge Type	Possible Span Length (ft)	Preferred Span Length (ft)
Precast pretensioned girder	30 to 200	30 to 180
Post-tensioned spliced girder	100 to 325	120 to 250
Precast segmental girder	200 to 450	250 to 400

The selection among these three bridge types is normally decided by span length requirements. As shown in Table 5.3.3-1, a single precast, pretensioned girder could be designed to span from 30 ft to 200 ft. Trucking length, crane capacity, and transporting routes may limit the girder length (and weight) that could be delivered. Therefore, a girder may need to be manufactured in two or more segments and shipped before being spliced together on-site to its full span length. Such splicing techniques can be applied by using post-tensioning systems for both single-span and multiple-span bridges, which span up to 325 ft. For span lengths over approximately 250 ft, precast segmental girder bridges may be considered, which is beyond the scope of this document. Section 5.3.3.3 further addresses spliced girder bridges.

5.3.3.1 Single-Span Bridges

As the simplest application of PC girders, single-span bridges normally consist of single girders. As shown in Figure 5.3.3-1, girders are set onto bearing pads at seat-type abutments. Dead and live load effects are based on a simply supported condition. PC girders obviously end themselves to being single-span elements because they are fabricated as single elements. Abutments can be seat-type or end diaphragm-type.

Figure 5.3.3-1 Single-Span I Beam Lowered onto Abutments at Mustang Wash Bridge (Bridge No. 54-1279L, Caltrans)

5.3.3.2 Multi-Span Bridges

Many design considerations for single-span bridges apply to multi-span bridges because girders or girder segments exist as single-span elements for several stages, namely, fabrication, transportation, erection, and deck pour. In addition, some multi-span bridges or portions thereof are constructed using expansion joints that can produce a simply supported condition for a span.

Most multi-span bridges are constructed with simple-span girders made continuous for live load to increase efficiency and redundancy. Limiting expansion joints, designing deck reinforcement to serve as negative moment reinforcement at interior bents, and providing girder continuity at bents can be achieved by using a continuous CIP deck and/or CIP diaphragms.

In addition, some bridges are detailed to provide an integral connection with full moment transfer between the superstructure and substructure. To achieve this, use CIP diaphragms at bent caps; reinforcing bars between the bent cap, diaphragm, and girders; and/or longitudinal post-tensioning. An integral connection provides not only longitudinal continuity for live load but also longitudinal continuity for seismic loading. Due to moment continuity between the superstructure and substructure, columns in multi-column bents may be designed to be pinned at their base, thus reducing foundation cost.

The following sections summarize three typical bent cap configurations for achieving continuity in multi-span bridges:

- Drop caps
- Inverted-tee caps
- Integral caps with precast post-tensioned girders

5.3.3.2.1 Drop Caps

Figure 5.3.3-2 Drop Cap at Chuckwalla Wash Bridge (Bridge No. 54-1278L, Caltrans)

Drop caps are bent caps that provide intermediate supports for girders together with live-load continuity (Figure 5.3.3-2). Drop caps are commonly detailed to provide a non-integral connection-without moment continuity to the substructure but with moment continuity in the superstructure through negative moment reinforcement in the deck. Simple-span girders are placed on bearing pads at the top of drop caps. Girders at the top of drop caps are normally tied together with a CIP diaphragm and dowels placed through the webs at the ends of the girders. As shown in Figure 5.3.3-3, steel pipe shear keys may extend from the top of the drop cap into the CIP diaphragms at bent caps. With pipe shear keys, moment transfer is prevented between the superstructure and substructure, and the bearing can more easily be replaced if needed.

With proper design and detailing of the diaphragm and bent cap, an integral connection can be developed between the superstructure and substructure, as shown in Figure 5.3.3-4. For example, the system can be designed to emulate seismic performance of a continuous CIP PT concrete bridge if the joint between girder and cap (due to positive moment during a seismic event) is prevented from opening. One method is to extend pretensioning strands through the joint for development within the cap, in accordance with the requirements of SSC 2.0. As mentioned in the subsequent section on integral caps with post-tensioned precast girders, post-tensioning of the girders to the cap at intermediate supports can also be used. The designer is encouraged to clearly detail the reinforcement between the superstructure, diaphragm, and bent cap so that conditions assumed in design realistically match field conditions.

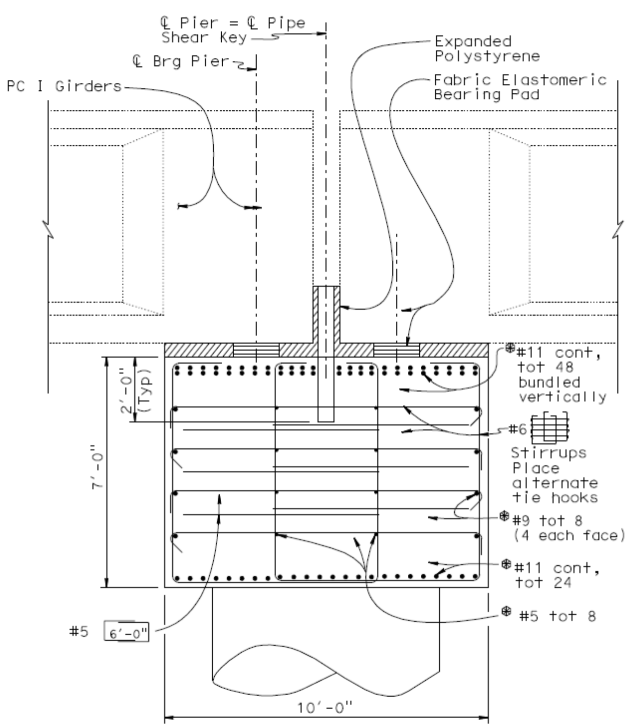


Figure 5.3.3-3 Nonintegral Drop Cap Example Detail Using Pipe Shear Key

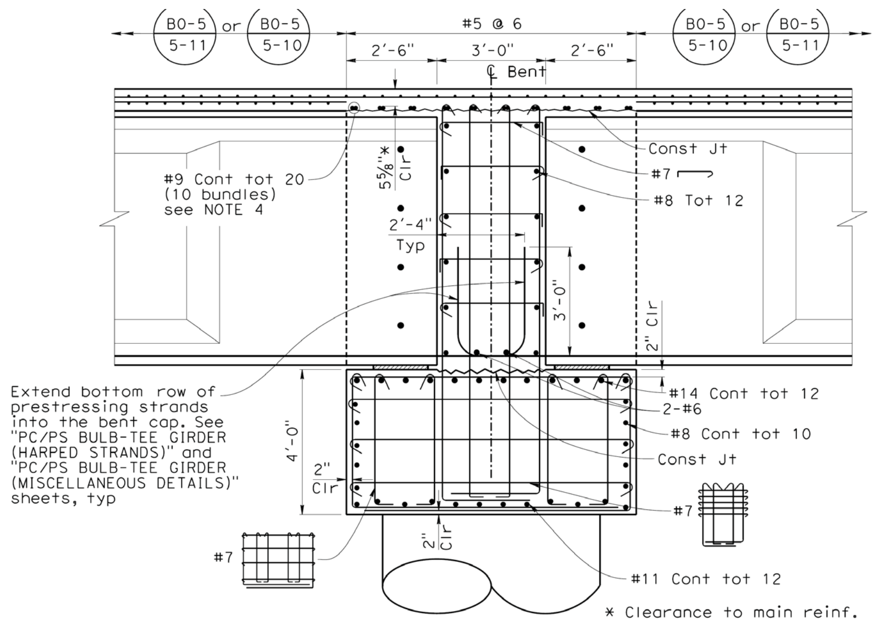


Figure 5.3.3-4 Integral Drop Cap Example Detail

Adequate seat width must be provided for drop caps to prevent unseating due to longitudinal displacement in a seismic event. Aesthetics should also be considered in the use of drop caps, as they lack the clean lines of inverted-tee caps or CIP PT box girders with integral caps.

5.3.3.2.2 Inverted-Tee Caps

Using an upside down "T" shaped cross section with a ledge, inverted-tee caps combine the ability to place precast girders directly on the bent and the aesthetic appeal of the flush bottom of cap with the precast girders. Hooked reinforcement extending from side faces of the cap is placed between girders, and a diaphragm is cast to tie the girders and cap together. A deck is later cast for live-load continuity. This is shown in Figures 5.3.3-5 and 5.3.3-6.

Designers have commonly modeled this connection as a pin (i.e., nonintegral connection between the superstructure and substructure) due to the assumption that the connection would degrade to a pin in a seismic event. However, recent research demonstrated that plastic hinges do indeed form at the column top, confirming that moment continuity develops due to the use of the CIP diaphragm and dowel bars through the girder webs (Snyder, 2010). For this connection type, continuity at the column top may be assumed, and joints may be designed for the force transfer associated with plastic hinging. Confining reinforcement at the column top is required.

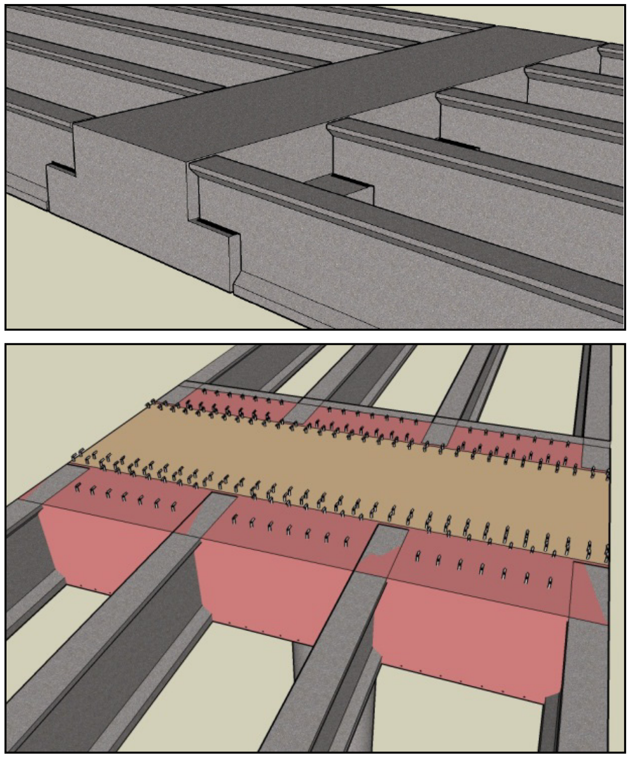


Figure 5.3.3-5 Dapped End Girder with Inverted-Tee Cap (Snyder, 2010)

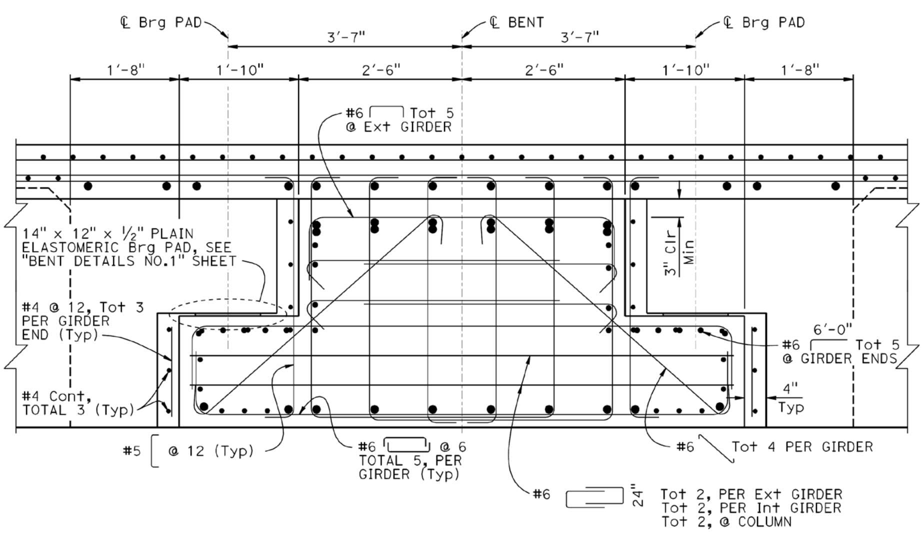


Figure 5.3.3-6 Existing Inverted-Tee to Dapped End Girder Connection Example Detail

5.3.3.2.3 Integral Caps with Precast Post-Tensioned Girders

Post-tensioning PC girders through a CIP bent creates an integral connection between the superstructure and substructure as well as a frame that is continuous for service, strength, and extreme event limit states (Figure 5.3.3-7). In addition, such a connection provides a means for bridge widening using PC girders to match the performance and appearance of an existing CIP PT bridge. Without an integral connection, continuity is not effectively developed at the bent cap, which would require columns and foundations to be designed to provide the necessary fixity at the base of the structure.

If the connection between post-tensioned PC girders and the bent cap is designed and detailed properly, the system can emulate the seismic performance of a continuous CIP PT concrete bridge (Holombo et al., 2000; Castrodale and White, 2004). Post-tensioning of the girders to the cap and intermediate supports is intended to prevent joint opening due to positive moment during a seismic event. Extending bottom pretensioning strands into the cap for development provides positive moment capacity.

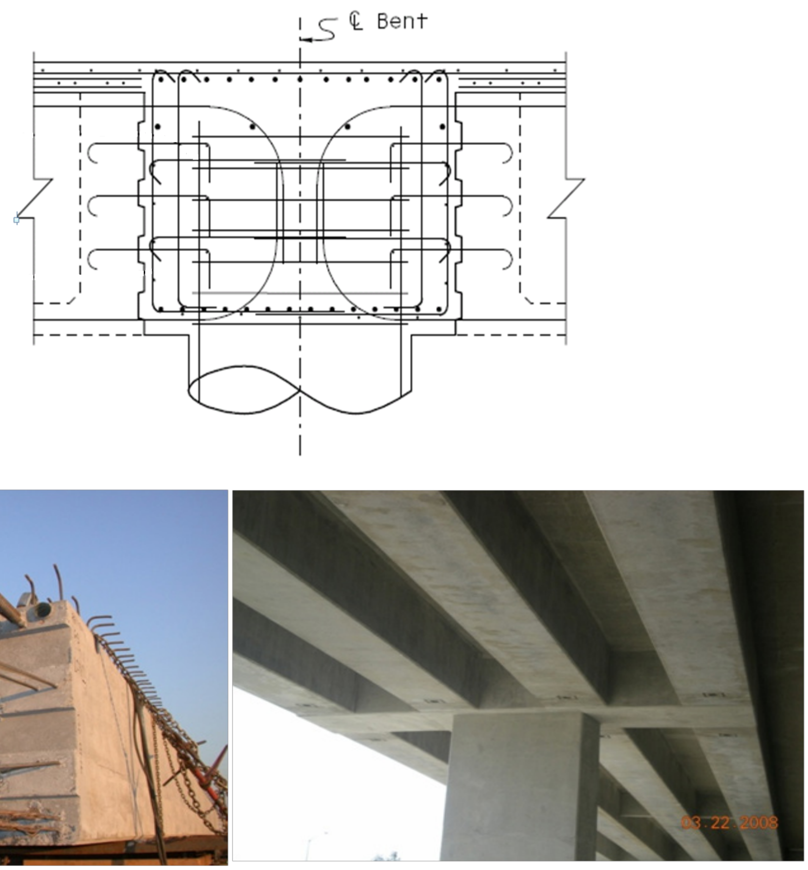


Figure 5.3.3-7 Integral Bent Cap Connection Using Longitudinal Post-tensioning of PC Girders

5.3.3.3 Spliced Girder Bridges

Due to limitations in transportation length and member weight, as well as stressing bed size, a girder may need to be fabricated in two or more segments and shipped before being spliced together on-site to its full span length. Such splicing techniques can be applied to both single-span and multiple-span bridges. By using this approach, the designer has significant flexibility in selecting the span length, number and location of intermediate supports, segment lengths, and splice locations. Splicing is more commonly used for multi-span bridge construction. However, spliced girders have also been used successfully in the construction of several single-span bridges in California such as the Angeles Crest Bridge (208 ft).

Splicing of girders is typically conducted on-site, either on the ground adjacent to or nearby the bridge location, or in place using temporary supports. Figure 5.3.3-8 shows two precast bath-tub girder segments being placed on temporary supports in preparation for field splicing at midspan.

Figure 5.3.3-8 Precast Bath-tub Girder Segments Spliced Near Midspan Using Temporary Supports at Harbor Blvd. Overcrossing (Bridge No. 22-0108, Caltrans)

Full continuity needs to be developed between spliced girder segments. This is commonly achieved using post-tensioning tendons between segments and mechanical coupling of reinforcement that is extended from the ends of the girder segments within a CIP closure pour. Figure 5.3.3-9 shows these details at the closure pour, including the use of couplers for PT ducts and ultimate butt splice couplers for reinforcement.

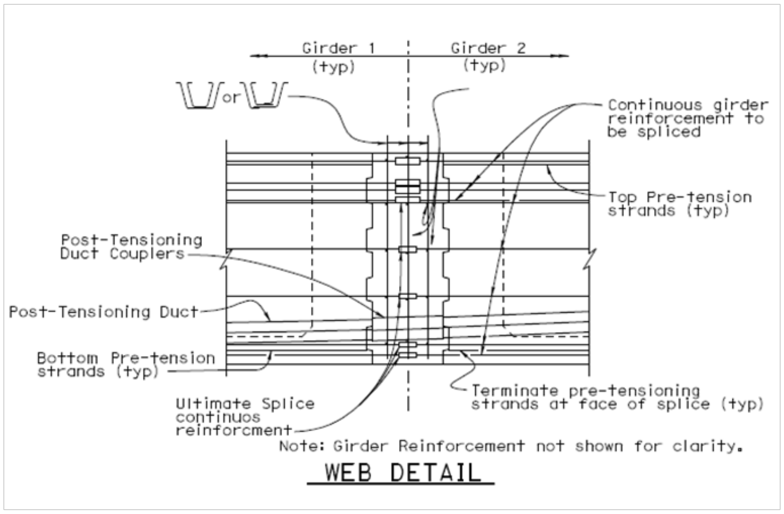


Figure 5.3.3-9 Details of Spliced Girder Closure Pour Using Mechanical Splices and PT Duct Couplers (Bridge No. 22-0108, Caltrans)

Post-tensioning spliced girders not only provides continuity but also enhances structural efficiency. Post-tensioning enhances interface shear capacity across the splice joint (closure pour), which normally includes roughened surfaces or shear keys (Figure 5.3.3-9).

When splicing together multiple spans of PC girders, it is critical that the precast girder placement, post-tensioning sequence, and material properties be properly defined. Figure 5.3.3-10 shows the construction sequence of a typical two-span (or multi-span) spliced girder bridge. At each stage, the following must be checked: concrete compressive strength and stiffness, creep and shrinkage of concrete, and tension force in the prestressing steel (and debonded length, if needed). The designer must consider each stage as the design of an individual bridge with given constraints and properties defined by the previous stage.

Design of the simplest multi-span precast spliced girder system includes consideration of a minimum of four stages or steps after fabrication and before service loads, as follows:

- Transportation: The girder acts as a simply supported beam, with supports defined by the locations used by the trucking company. Typically, the manufacturer or trucking company is responsible for design and check of loads, stability, and bracing during transportation and erection of the girder.
- Erection: The girder initially acts as a simply supported beam, with supports defined by the abutments, bents or temporary falsework locations. A CIP closure pour is placed after coupling of PT tendons and reinforcing bars in the splice joint.

  Note: First stage of two-staged post-tensioning may be applied before the deck pour instead of after the deck pour based on design (not shown in Figure 5.3.3-10).
- Deck pour: The deck is poured but not composite with the girders until attaining full strength. Therefore, the girders alone carry girder self-weight and the wet deck weight.

- Post-tensioning: The hardened deck and girder act compositely, and the girders are spliced together longitudinally using post-tensioning. As the number of girders that are spliced and the stages of post-tensioning increases, so does the complexity of design.

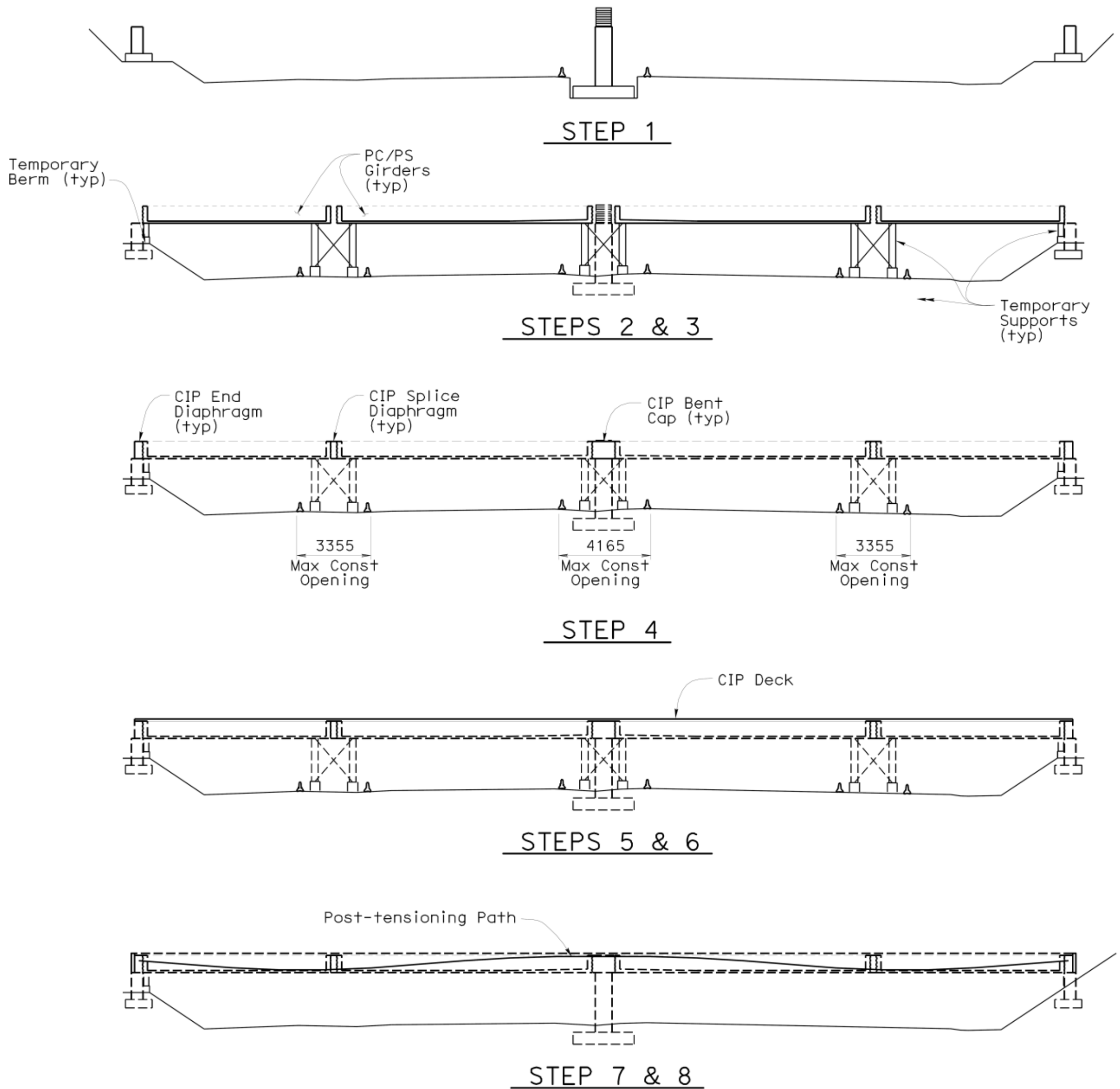


Figure 5.3.3-10 Spliced Bridge Construction Sequence Example (Bridge No. 22-0108, Caltrans)

5.3.4 DESIGN CONSIDERATIONS

PC girder design must address three basic stages−transfer, service, and ultimate−as well as additional stages if post-tensioning is introduced. PC girder design, including section size, prestress force (number and size of strands), strand layout, and material properties, may be governed by any of these stages. Although design for flexure dominates the PC girder design process, other aspects must also be considered, such as prestress losses, girder shear and interface shear strength, deflection and camber, anchorage zones, diaphragms, and end blocks. The following sections briefly introduce the various aspects of PC girder design.

The designer is encouraged to read the references cited in the following sections, particularly AASHTO-CA BDS-8, Caltrans SDC 2.0, BDM 5.3.

5.3.4.1 Materials

5.3.4.1.1 Concrete

Concrete used in PC girders produced under plant-controlled conditions is typically of higher strength and higher quality than for CIP concrete. Per BDM 5.3, the minimum concrete compressive strength at release, f'_{ci}, and minimum 28-day concrete compressive strength, f'_c, for PC girders is 4 ksi. In addition, the concrete compressive strength at release, f'_{ci}, may be selected as large as 7 ksi and f'_c as large as 10 ksi. However, designers should verify with local fabricators' economical ranges of f'_{ci} on a project-specific basis, especially for f'_{ci} and f'_c exceeding these limits. Minimum concrete compressive strengths may also be specified at girder erection and for post tensioning, when used.

In most PC girder design, a relatively large value of f'_{ci} is used in design, which typically controls the overall concrete mix design. If an excessively large value of f'_{ci} is required in design to resist temporary tensile stresses at transfer in areas other than the precompressed tensile zone, such as the top flange at girder ends, then bonded reinforcement or prestress strands may be designed to resist the tensile force in the concrete, per stress limits in LRFD Specifications Table 5.9.2.3.1b-1 (AASHTO, 2017). This helps reduce the required f'_{ci} used in design.

The relatively large value of f'_{ci} used in design also results in a relatively large value of f'_c (e.g., often in excess of 7 ksi), which is normally larger than that required to satisfy the concrete compressive strength requirements at the serviceability and/or ultimate limit state. In cases where a larger f'_{ci} is required to produce an economical design (e.g., girders of long span, shallow depth, or wide spacing), high strength concrete mixes that require longer than the normal 28-day period may be specified. Current Standard Specifications (Caltrans, 2018) allow 42 days for achieving specified strength and 56 days for low cement mixes. However, designers should verify the impact of such a decision on the overall construction schedule.

Advantages of the concrete used in PC girders produced under plant-controlled conditions are wide ranging. Higher modulus of elasticity and lower creep, shrinkage, and permeability are by-products of the relatively higher compressive strength and steam curing process used for PC girders. In addition, reduced effects of creep and shrinkage for PC girders occur after installation because most creep and shrinkage occurs prior to erection. Supplementary cementitious materials (SCMs) and regional materials may also be used for benefits in cost, material properties, and environmental impact through the use of in-house batch plants, mix designs, and sustainability practices.

Self-consolidating concrete (SCC), a highly flowable yet cohesive concrete that consolidates under its own weight, is becoming more commonly used in precast plants. It provides significant advantages such as elimination of external and internal vibration for consolidation and reduced manual labor and equipment requirements resulting in reduced construction time; excellent consolidation, even in congested regions of reinforcement; higher level of quality control; extremely smooth concrete surfaces, even in negative draft regions; eliminated need for patching; increased safety; and lower noise levels, usually combined with higher strength and improved durability. Some disadvantages of SCC include more costly material, stricter control on selection and measurement of materials, larger number of trial batches, greater sensitivity to water content, more rapid hardening, faster drying, higher formwork design loads (for fluid pressure), as well as greater experience and care in handling and production of SCC.

5.3.4.1.2 Steel

For economy, PC girders commonly use 0.6 in. diameter, 270 ksi (Grade 270), low-relaxation strands. Use of 0.5 in. diameter strands is less common because the 0.6 in. diameter strands provide a significantly higher efficiency due to a 42% increase in capacity. However, 0.375 in. diameter strands are commonly used for stay-in-place, precast deck panels. If epoxy coated prestressing strands are required, a note should be shown on the design plans, and the corresponding section of the Standard Specifications should be used.

Deformed welded wire reinforcement (WWR), conforming to ASTM A1064 and Caltrans Standard Specifications based on a maximum tensile strength of 60 ksi, is permitted and commonly used as shear reinforcement in PC girder design.

5.3.4.2 Prestress Losses

From the time prestressing strands are initially stressed, they undergo changes in stress that must be accounted for in design. Figure 5.3.4-1 illustrates the change in strand stress over time for a typical pretensioned girder.

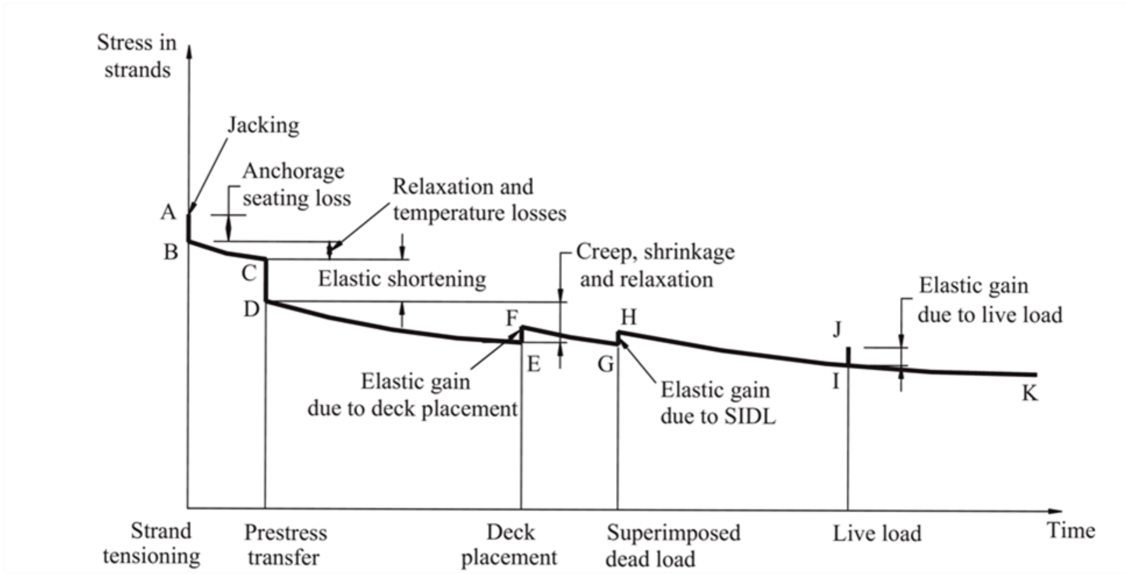


Figure 5.3.4-1 Strand Stress vs. Time in Pretensioned Girder (Tadros et al., 2003)

Prestress losses in prestressed concrete members consist of instantaneous (or immediate) and time-dependent losses in prestressing strands. Total losses can be estimated using the AASHTO-CA BDS-8 approach:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \qquad \text{(AASHTO 5.9.3.1-1)}$$

where:

Δf_{pT} = total change in stress due to losses (ksi)

Δf_{pES} = sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads (ksi)

Δf_{pLT} = losses due to long-term shrinkage and creep of concrete, and relaxation of the steel (ksi)

Losses are normally defined from the time of initial stress (immediately after seating of strands for PC girders). Time-dependent losses of prestress include concrete creep and shrinkage and steel relaxation. AASHTO-CA BDS-8 provides an approximate estimate and refined estimate for determining time-dependent losses. The background can be found in the *National Cooperative Highway Research Program (NCHRP)* Report 496*, Prestress Losses in Pretensioned High-Strength Concrete Bridge Girders* (Tadros et al., 2003).

For PC girders, instantaneous loss refers to loss of prestress due to elastic shortening of the girder at transfer. Elastic gain refers to increase in strand stress due to strand extension related to application of external loads.

A reasonable estimate of prestress losses is critical to properly estimate the required prestress force (and thus the required number of strands). Overestimating losses leads to a larger than necessary initial prestress force, which results in larger initial tensile and compressive stresses and may, in turn, result in cracking and larger than expected camber. Overestimation of losses tends to reduce design efficiency because of the increase in number of strands, f'_{ci} cost of the concrete mix, and/or curing time. In addition, problems in girder placement and haunch height in the field may result from excessive camber. Although underestimating losses could potentially produce adverse effects such as flexural cracking in the precompressed tensile zone at service level, such problems have rarely been found to occur in practice.

5.3.4.2.1 Instantaneous Losses

In PC girders, the entire prestressing force is applied to the concrete in a single operation. For pretensioned members, the loss due to elastic shortening can be calculated from AASHTO Eq. 5.9.3.2.3a-1, as shown below:

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp} \qquad \text{(AASHTO 5.9.3.2.3a-1)}$$

where:

Δf_{pES} = sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads (ksi)

f_{cgp} = the concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi)

E_p = modulus of elasticity of prestressing steel (ksi)

E_{ct} = modulus of elasticity of concrete at transfer or time of load application (ksi)

Calculation of Δf_{pES} requires iteration for f_{cgp}. However, iteration can be avoided by using AASHTO Eq. C5.9.3.2.3a-1 (AASHTO, 2017) for Δf_{pES}. It is important that Articles C5.9.3.2.3a and C5.9.3.3 be consulted when using transformed section properties in the stress analysis.

5.3.4.2.2 Time-Dependent Losses

AASHTO-CA BDS-8 provides two methods to estimate the time-dependent prestress losses: approximate method (Article 5.9.3.3) and refined method (Article 5.9.3.4). This chapter introduces a sample calculation using the approximate method. However, for cases in which the refined method is required or preferred, the designer should follow Article 5.9.3.4. Chapter 9 of the PCI Bridge Design Manual (2014) provides useful PC girder design examples with prestress loss calculations using both the refined and approximate methods.

Per Article 5.9.3.3, the approximate method is applicable to standard precast, pretensioned members subject to normal loading and environmental conditions, where:

- Members are made from normal-weight concrete
- Concrete is either steam- or moist-cured
- Prestressing strands use low relaxation properties
- Average exposure conditions and temperatures characterize the site

In addition, the estimate is intended for sections with composite decks. This method should not be used for uncommon shapes (volume-to-surface ratios, *V*/*S,* significantly different than 3.5 in.), unusual level of prestressing, or with complex construction staging.

Long-term prestress losses due to creep and shrinkage of concrete and relaxation of steel are estimated using the following formula, in which the three terms corresponds to creep, shrinkage, and relaxation, respectively:

$$\Delta f_{pLT} = 10\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12\gamma_h\gamma_{st} + \Delta f_{pR} \qquad \text{(AASHTO 5.9.3.3-1)}$$

where:

A_g = gross area of girder section

A_{ps} = area of prestressing steel

f_{pi} = prestressing steel stress immediately prior to transfer (ksi)

H = average annual ambient mean relative humidity (percent)

γ_h = correction factor for relative humidity of ambient air

= $1.7\text{-}0.01H$

γ_{st} = correction factor for specified concrete strength time at of prestress transfer to concrete member

= $5/(1+ f'_{ci})$

Δf_{pR} = an estimation of relaxation loss taken as 2.4 ksi for low relaxation strand, 10 ksi for stress relieved strand, and in accordance with manufacturers' recommendation for other types of strand (ksi)

5.3.4.3 Flexure

Section 5.3.2 provides a detailed summary of flexural stress design provisions, with limit states for service (including transfer), strength, and fatigue in accordance with AASHTO-CA BDS-8. Figures 5.3.2-3 through 5.3.2-6 illustrate the change in flexural stress distribution near midspan for a typical PC girder at transfer, deck pour, and service level. Nominal flexural resistance of PC girders shall be in accordance with Articles 5.6.3.1 and 5.6.3.2.

BDM 5.3 provides specific guidance for design of PC girders, addressing issues such as:

- Order of design (service limit state followed by strength check)
- Live load continuity and negative moment reinforcement over the bents
- Determination of P_j and centroid of PS steel (CGS) and their inclusion on plan sheets
- Harping versus debonding, including tolerances for harping and debonding provisions
- Use of temporary strands and associated blockouts
- Positive moment reinforcement for continuous spans
- Design modifications for long span girders

In addition, the following practical aspects should also be noted in carrying out flexural design of PC girders:

- The initial girder section size is typically based on the minimum depth-to-span ratio required for a given girder type.
- The specified concrete compressive strengths (initial and 28-day) are commonly governed by the initial compressive strength, f'_{ci}, required to limit stresses at transfer.
- The total prestress force (number and size of strands) and strand layout are usually determined to satisfy the service limit state (Service III) but may have to be revised to satisfy flexural strength at ultimate (Strength II, California P-15 permit truck).
- Girder design is based on the minimum overall depth when computing capacity of the section.

5.3.4.4 Shear

5.3.4.4.1 Shear Design for Girders

Per BDM 5.3, shear design of PC girders is performed using the sectional method specified in Article 5.7.3 (AASHTO, 2017). The sectional method is based on the Modified Compression Field Theory (MCFT), which provides a unified approach for shear design for both prestressed and reinforced concrete components (Collins and Mitchell, 1991). The MCFT is based on a variable angle truss model in which the diagonal compression field angle varies continuously, rather than being fixed at 45˚ as assumed in prior codes. For prestressed girders, the compression field angle for design is typically in the range of 20˚ to 40˚.

Per Article C5.7.3.4, simplified shear design procedure as specified in Article 5.7.3.4.1 cannot be used in PC girder design.

For disturbed regions, such as those occurring at dapped ends, shear provisions using the strut and tie method should be used (AASHTO, 2017).

In the sectional method, a component is investigated by comparing the factored shear force and the factored shear resistance at a number of sections along the member length. Usually this check is made at a minimum of tenth points along the span as well as at locations near the supports.

Because shear design typically follows flexural design, certain benefits can be realized in shear design. For example, when harped strands are used, the vertical component of the harped strand force contributes to shear resistance. In addition, the higher strength concrete required for flexure enhances the V_c term for shear design. Because flexure-shear interaction must be checked per Article 5.7.3.5 (AASHTO, 2017), the longitudinal reinforcement−based on flexural design must be checked after shear design, to ensure that sufficient longitudinal reinforcement is provided to resist not only flexure (and any axial forces along the member), but also the horizontal component of a diagonal compression strut that generates a requirement for longitudinal reinforcement. Article 5.7.3.3 includes an upper limit on the nominal shear resistance, V_n, that is independent of transverse reinforcement, to prevent web crushing prior to yielding of transverse reinforcement.

For skewed bridges, live load shear forces in the exterior girder of an obtuse angle must be magnified in accordance with Article 4.6.2.2.3c unless a three-dimensional skewed model is used.

To accommodate field bending of stirrups, #4 or #5 stirrups are commonly preferred. In most cases, the size of stirrups should not exceed #6.

5.3.4.4.2 Interface Shear Design

Interface shear should be designed based on the shear friction provisions of Article 5.7.4 and BDM 5.3.

5.3.4.5 Deflection and Camber

5.3.4.5.1 Key Aspects for Design

Designers must address potentially challenging issues related to downward deflection and upward camber of PC girders. Camber in a PC girder occurs instantaneously at transfer but can increase to much larger values long-term, particularly due to creep and shrinkage of the concrete. Excessive camber at erection may cause potential intrusion of the top flange of the girder into the CIP deck. Although the contractor is responsible for deflection and camber calculations (per Caltrans Standard Specifications and BDM 5.3), the designer is responsible for specifying a midspan haunch thickness and calculating the minimum haunch thickness at supports, which affects the total bridge depth at both mid-span and at supports. In order to calculate the minimum haunch thickness at supports, girder deflections at release and at erection, as well as immediate girder deflection due to the deck weight, must be considered. To complete the deflection design and provide better construction support, the following guidelines are recommended:

- Specify unfactored instantaneous girder deflections on plan sheets: Per Caltrans Standard Specifications, the contractor is responsible for deflection and camber calculations and any required adjustments for deck concrete placement to satisfy minimum vertical clearance, deck profile grades, and cross slope requirements. However, the designer must provide, on plan sheets, the unfactored instantaneous girder deflections due to:
  - Deck and haunch weight on the non-composite girder
  - Weight of barrier rail and future wearing surface on the composite girder-deck section

  These deflection components are used to set screed grades in the field. For spliced girders, instantaneous upward deflections due to post-tensioning at different stages should be shown on the design plans.

- Determine minimum haunch thickness and specify on plan sheets: The haunch is the layer of concrete placed between the top flange of the girder and bottom of deck to ensure proper bearing. It accommodates construction tolerances such as unknown camber of the girder at time of erection. Because camber values vary along the span length, the actual haunch thickness varies along the span, too. The designer should specify the haunch thickness at mid-span and then calculate the minimum required haunch thickness at supports.

  The haunch:
  - Accommodates variation in actual camber
  - Allows the contractor to adjust screed grades
  - Eliminates potential intrusion of the top flange of the girder into the CIP deck
  - Establishes the seat elevation at supports

  Cross slope and width at the top flange of the girder should be considered in determining the specified midspan haunch thickness.

  The typical section should show:
  - Minimum structure depth at centerline of bearing at the supports, including girder depth, deck thickness, plus calculated haunch thickness
  - Minimum structure depth at mid-span, including girder depth, deck thickness, plus any haunch thickness the designer specifies

It should be noted that for girders with large flange widths, such as the CA wide-flange girder, a larger haunch thickness might add a significant concrete quantity and weight to the design.

- Satisfy LRFD Specifications for live load deflection: Service level deflections may be checked per Article 2.5.2.6.2, which suggests a limit of $L/800$ for live load deflection due to HL-93 vehicular loading. This is an optional check and not required per LRFD Specifications. Because this is an instantaneous deflection check, no multipliers for long-term deflection should be used. The modulus of elasticity should be determined based on AASHTO Eq. 5.4.2.4-1 (AASHTO, 2017) and the effective moment of inertia, I_e, should be used per Article 5.6.3.5.2.

- Verify girder camber is controlled at key stages: The designer may work with the construction structure representative to ensure that the estimated PC girder camber and camber growth are controlled throughout all key stages, such as fabrication, erection, deck placement, and service level. Camber should not be excessive (i.e., causing concern over intrusion of the top flange of the girder into the CIP deck) and should be positive (upward) under both short-term and long-term conditions. This requires the designer to be aware of girder deflection due to prestress force and dead loads, as well as the timing of their application. This can be especially important for bridge widenings. When more accurate camber values are required for unusual cases such as widening of a long span bridge, the assumed age of the girder at various stages may need to be shown on plan sheets.

5.3.4.5.2 Calculation Approaches

Total deflection of a girder at any stage is the sum of the short-term and long-term deflections. Short-term deflections are immediate deflections based on the modulus of elasticity and effective moment of inertia of the appropriate section. Some loads (such as girder and deck self-weight) are carried by the precast girder alone, while other loads are carried by the much stiffer composite girder-deck system (such as barriers, overlays, as well as live loads). Long-term deflections consist of long-term deflections at erection and long-term deflection at final stage (may be assumed to be approximately 20 years). Long-term deflections at erection are more coarsely determined because of the highly variable effects of creep and shrinkage. Therefore, although theoretical values and various procedures to determine instantaneous and long-term camber and deflection of PC girders are available, calculated values must be viewed as merely estimates.

Table 5.3.4-1 lists common equations for instantaneous camber of PC girders for different prestress configurations. Long-term deflections at erection and final stage are typically estimated based on one of three approaches:

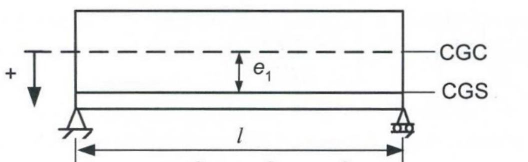
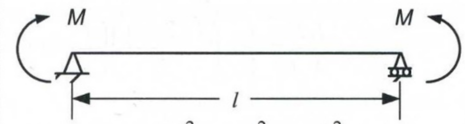
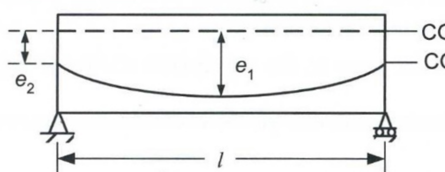
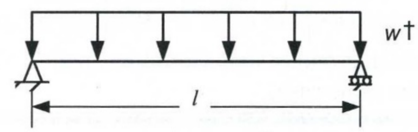
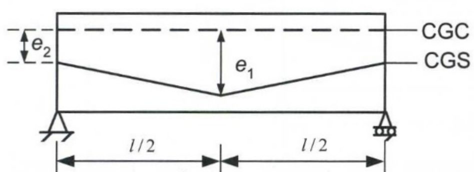
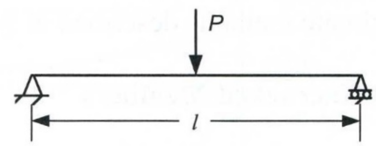
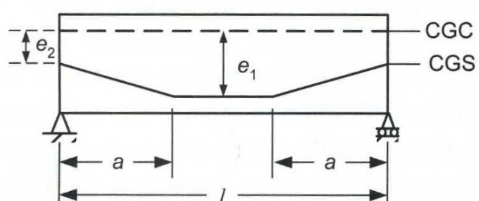
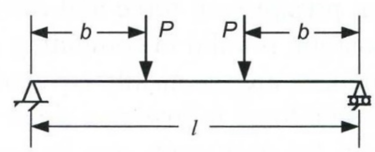
- Historic multipliers shown in Table 5.4.3.-2 (PCI 2014)
- Modified multipliers based on regional industry experience
- Detailed time-step analysis accounting for various construction stages and varying material properties

Use of multipliers (either historic or regionally modified) for girders is the most common approach for estimating long-term deflections at erection of routine bridges in California. The design example in Section 5.3.6 uses the historic multiplier method. Instantaneous deflection due to prestressing force and girder weight is calculated at release. Long-term deflection of precast concrete girders at erection is then calculated as the instantaneous deflection multiplied by a multiplier. In performing calculations, camber due to prestressing force and the self-weight of girder, as well as deflections due to the weight of deck and haunch are calculated using the initial modulus of elasticity of concrete and section properties for the non-composite girder. Then, deflections due to the concrete barrier and future wearing surface are calculated using gross composite section properties.

The historic multiplier method is a simple and straightforward method. Even though it is found to give reasonably accurate prediction of the deflection at time of erection, it is not recommended for estimating long-term deflection of bridges comprised of beams that are made composite with a cast-in-place deck slab. This method does not account for the relatively significant effects of cast-in-place concrete deck, as described here. Once the deck is hardened, it restrains the beam from creeping upward (due to prestressing). In addition, the differential creep and shrinkage between girders and cast-in-place concrete deck results in changes of the bridge member deformation.

The design example in Section 5.3.6 illustrates the use of Table 5.3.4-2 to estimate long-term camber and deflection to determine minimum required haunch thickness at supports. Chapter 9 of the PCI Bridge Design Manual (2014) provides additional example calculations for camber and deflection.

Table 5.3.4-1 Camber and Rotation Values for Various Prestress Configurations (Naaman, 2004)

Camber due to prestressing force	Deflection due to loading
 $\Delta = -\frac{Fe_1 l^2}{8EI} = \Phi_1 \frac{l^2}{8} = \Phi_2 \frac{l^2}{8}$ → (see footnote)	 $\Delta = \frac{Ml^2}{8EI} = \Phi_1 \frac{l^2}{8} = \Phi_2 \frac{l^2}{8}$
 $\Delta = -\frac{Fl^2}{8EI}\left[e_2 + \frac{5}{6}(e_1 - e_2)\right] = \Phi_1 \frac{l^2}{8} + (\Phi_2 - \Phi_1)\frac{l^2}{48}$	 $\Delta = \frac{5wl^4}{384EI} = \Phi_1 \frac{5l^2}{48}$ † Assumed uniform per unit length
 $\Delta = -\frac{Fl^2}{24EI}[2e_1 + e_2] = \Phi_1 \frac{l^2}{8} + (\Phi_2 - \Phi_1)\frac{l^2}{24} = \Phi_1 \frac{l^2}{12} + \Phi_2 \frac{l^2}{24}$	 $\Delta = \frac{Pl^3}{48EI} = \Phi_1 \frac{l^2}{12}$
 $\Delta = -\frac{Fl^2}{8EI}\left[e_1 + (e_2 - e_1)\frac{4a^2}{3l^2}\right] = \Phi_1 \frac{l^2}{8} + (\Phi_2 - \Phi_1)\frac{a^2}{6}$	 $\Delta = -\frac{Pb}{24EI}\left(3l^2 - 4b^2\right) = \Phi_1 \frac{3l^2 - 4b^2}{24}$
For the uncracked section, use I of transformed section or I_{gross} as first approximation. For the cracked section, use I_e = effective moment of inertia.	

Note: For a straight tendon profile with debonding over a distance a from each support, the camber due to prestressing is given by: $\Delta = -(Fe_1/EI)(l^2/8 - a^2/2) = \Phi_1(l^2/8 - a^2/2)$

Table 5.3.4-2 PCI-Recommended Multipliers for Estimating Long-term Camber and Deflection for Typical PC Members (PCI, 2014)

	At Erection	Without Composite Topping	With Composite Topping
(1)	Deflection (↓) component: Apply to the elastic deflection due to the member weight at transfer of prestress	1.85	1.85
(2)	Camber (↑) component: Apply to the elastic camber due to prestress at the time of transfer of prestress	1.8	1.8
	Final		
(3)	Deflection (↓) component: Apply to the elastic deflection due to the member weight at transfer of prestress	2.7	2.4
(4)	Camber (↑) component: Apply to the elastic camber due to prestress at the time of transfer of prestress	2.45	2.2
(5)	Deflection (↓) component: Apply to the elastic deflection due to superimposed dead load only	3	3
(6)	Deflection (↓) component: Apply to the elastic Deflection caused by the composite topping	---	2.3

5.3.4.6 Anchorage Zones

5.3.4.6.1 Splitting Resistance

End splitting can occur along prestressing strands due to local bursting stresses in the pretensioned anchorage zone. To prevent failure, Article 5.9.4.4 requires vertical reinforcement, A_s, to be provided within a distance $h/4$ from the end of the girder to provide splitting or bursting resistance given by the following equation:

$$P_r = f_s A_s \qquad \text{(AASHTO 5.9.4.4.1-1)}$$

where:

A_s = total area of vertical reinforcement located within the distance $h/4$ from end of beam (in.^{2})

f_s = stress in steel not to exceed 20 ksi

P_r = factored bursting resistance of pretensioned anchorage zone provided by transverse reinforcement (kip)

h = overall dimension of precast member in the direction in which splitting resistance is being evaluated (in.)

Per Article 5.9.4.4.1, Pr should not be taken as less than 4% of the total prestressing force at transfer.

For spliced precast girders where post-tensioning is directly applied to the girder end block, general zone reinforcement is required at the end block of the anchorage area based on Article 5.9.5.6.

5.3.4.6.2 Confinement Reinforcement

Article 5.9.4.4.2 requires reinforcement be placed to confine the prestressing steel in the bottom flange, over the distance 1.5*d* from the end of the girder, using #3 rebar or larger with spacing not to exceed 6 in. and shaped to enclose the strands.

5.3.4.7 Diaphragms and End Blocks

Although intermediate diaphragms may not be required per Article 5.12.4, Caltrans practice and BDM 5.3 specify the use of one or more intermediate diaphragms for girders longer than 80 ft to improve distribution of loads between girders and to help stabilize the girders during construction. Also, per Article 5.12.4, end diaphragms are required at abutments, piers, and hinge joints. Due to increase in fabrication inefficiencies, girder weight, and overall cost, end blocks should only be used where essential for shear resistance. For more information, see BDM 5.3.

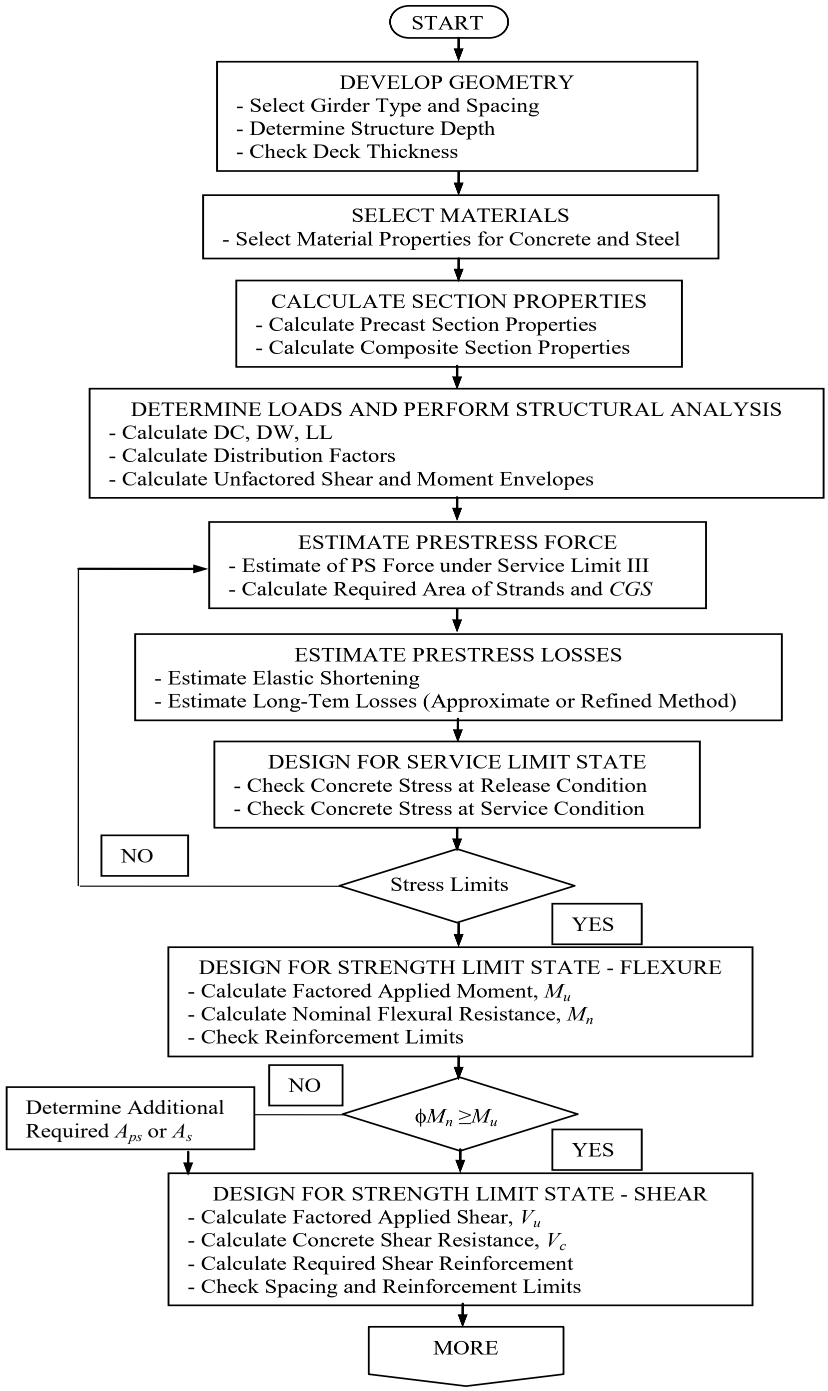
5.3.4.8 Lateral Stability

Because PC girders tend to be rather long slender members, they should be checked for lateral stability during all construction stages, including handling, transportation, and erection. Fabricators are normally responsible for all girder stability checks. However, the designer is encouraged to consider and verify lateral stability during design, especially when non-standard girders are selected.

Procedures for checking lateral stability were developed by Mast, 1993, and summarized in Section 8.10 of the PCI Bridge Design Manual (PCI, 2014). Some commercial software incorporates this method. The designer should verify specific assumed support and stability parameters (e.g., support locations, impact, transport stiffness, super elevation, height of girder center of gravity and roll center above road, and transverse distance between centerline of girder and center of dual tire) with local fabricators, contractors, and other engineers, as appropriate.

5.3.5 DESIGN FLOW CHART

The following flow chart shows the typical steps for designing single-span precast, prestressed concrete girders. The example in the next section closely follows this flow chart.



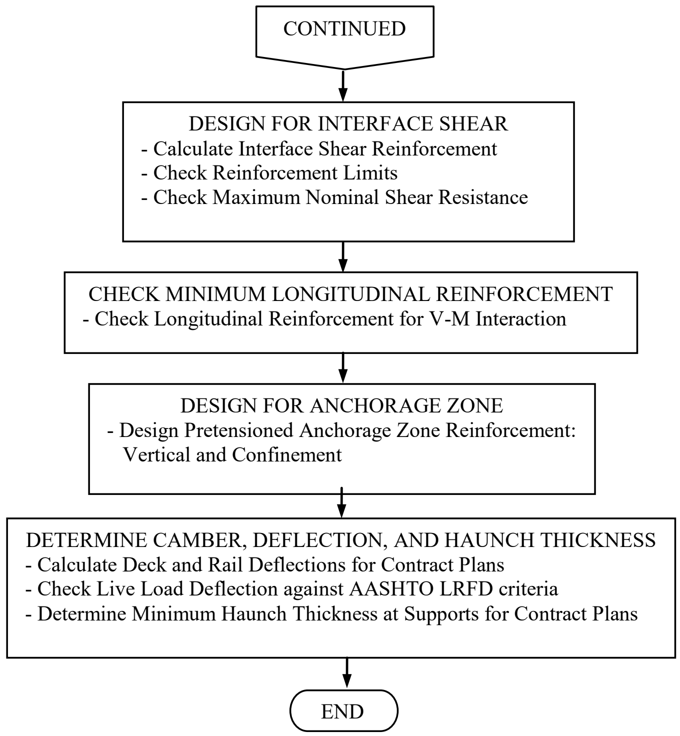


Figure 5.3.5-1 Precast/Pretensioned Concrete Girder Design Flow Chart

5.3.6 DESIGN EXAMPLE-PC SIMPLE SPAN I-GIRDER BRIDGE

This example illustrates the design procedure for a typical PC I-girder using the AASHTO-CA BDS-8 (AASHTO, 2017; Caltrans, 2019a).

To demonstrate the process, a typical interior girder of a 70 ft single-span bridge with no skew is designed using a standard California PC I-girder with composite CIP deck to resist flexure and shear due to dead and live loads. The design live load used for service limit design (Service I and III) is the HL-93 design truck, and the Caltrans P15 design truck is used for the strength limit design (Strength II). Elastic flexural stresses for initial and final service limit checks are based on transformed sections. AASHTO-CA BDS-8 Approximate Method is used to estimate long-term, time-dependent prestress losses based on gross section properties. Shear design is performed using the sectional method.

Major design steps include establishing structural geometry, selecting girder type and spacing, selecting materials, performing structural analysis, estimating prestress force, estimating prestress losses, service limit state design, strength limit state design, shear design, anchorage zone design, determining girder deflections and determining minimum haunch thickness at supports.

5.3.6.1 Problem Statement

A 70 ft simple-span bridge is proposed to carry highway traffic across a river. Preliminary studies have resulted in the selection of a PC concrete I-girder bridge based on traffic and environmental constraints at the site. Figures 5.3.6-1 and 5.3.6-2 show the elevation and plan views of the bridge, respectively. The span length (from centerline of bearing to centerline of bearing) is 70 ft and the girder length is 71 ft.

The required bridge deck width is 35 ft, which includes a 32 ft roadway and two 1.5 ft concrete barriers (type 736 was assumed. Need to use type 836 for future bridges). Three inches of polyester concrete overlay are assumed to be placed on the bridge as a future wearing surface (additional dead load on girders).

Design of a typical interior girder must satisfy all requirements of AASHTO-CA BDS-8 (AASHTO, 2017; Caltrans, 2019a) for all limit states.

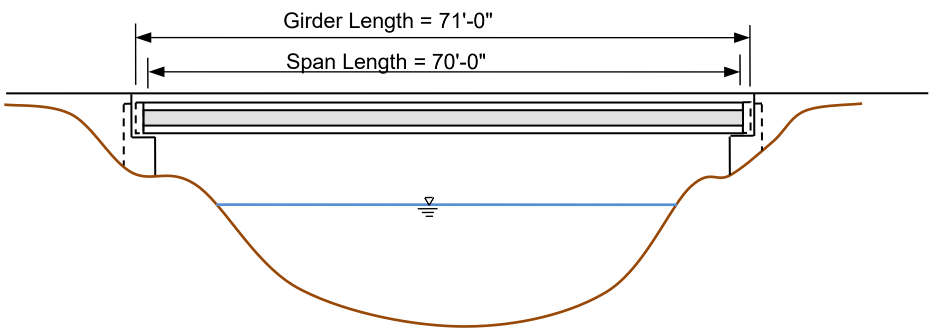


Figure 5.3.6-1 Elevation View of the Example Bridge

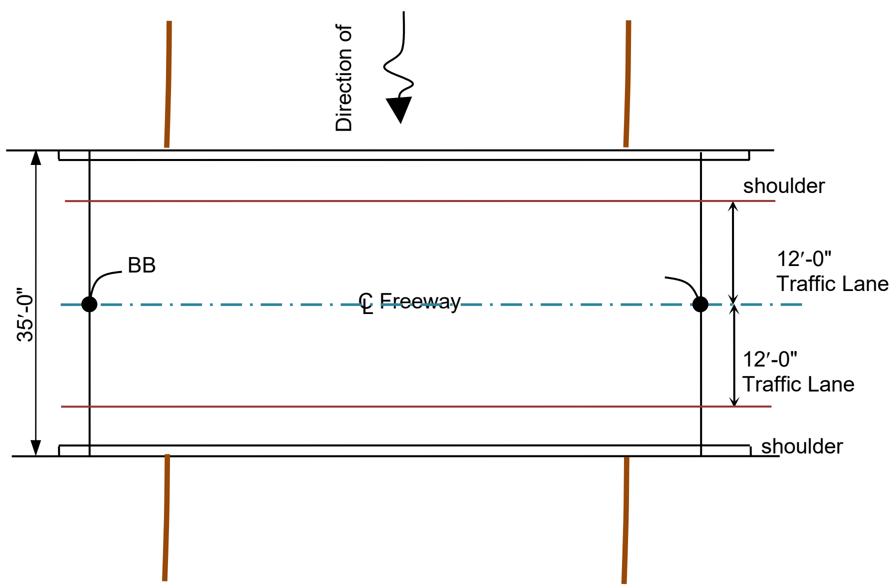


Figure 5.3.6-2 Plan View of Example Bridge

5.3.6.2 Select Girder Depth, Type, and Spacing

For a 70 ft span, the standard California I girder section has been found to be an efficient section, with a minimum structure depth-to-span length ratio (D/L) of 0.055 for simple spans. Also for PC girders, a girder spacing-to-structure depth ratio (S/D) of 1.5 is commonly used.

Span length, L = 70 ft

Assuming:

$$\frac{\text{Structure Depth}, D_s}{\text{Span Length}, L} = 0.055$$

The minimum depth is: D_s = 0.055 (70) = 3.85 ft

Because the deck thickness is based on girder spacing and girder spacing is based on structure depth, the concrete slab thickness must be initially assumed. Assume a slab thickness of 7 in. and later verify this value using Deck Slab Thickness and Reinforcement Schedule in BDM 9.4 after the girder spacing has been determined. (Note: BDM 9.4 was published in October 2021 after this example has been done. Table 9.4.5.2 of BDM 9.4 shows 8 in as the minimum deck thickness. For illustration purpose, this example assumes and uses 7 in as the deck thickness.)

Therefore, the minimum girder height = 3.85 (12) – 7 = 39.2 in.

Select a 42 in. standard California I girder (CA I42) from BDM 5.3 slightly larger than the minimum height.

Assuming a haunch thickness, t_h = 1 in. at midspan:

The structure depth, D_s = 42 + 1 + 7 = 50 in. (4.17 ft)

$$\frac{D_s}{L} = \frac{4.17}{70} = 0.060 > 0.055 \qquad \text{(OK)}$$

The center-center girder spacing is determined as follows:

Maximum girder spacing, $S = 1.5\ D_s$ = 1.5 (4.17 ft) = 6.26 ft

Total bridge width = 35 ft (assumed)

Try a girder spacing, S = 6 ft

$$\text{Overhang length} = \frac{35 - 6(5\text{ spacings})}{2\text{ overhangs}} = 2.5\text{ ft}$$

According to BDM 9.4 Figure 9.4.5.1, overhangs should be less than half the girder spacing (*S*/2) or 6 ft maximum.

$$\frac{2.5}{6} = 0.42\text{ ft} < 0.50\text{ ft (OK)}$$

Therefore, use 6 ft girder spacing.

Determine deck thickness:

From BDM 9.4 Deck Slab Thickness and Reinforcement Schedule, for girder centerline-to-centerline spacing of 6 ft, the required slab thickness is 7 in. Therefore, a 7 in. deck thickness can be used.

The established typical cross section of the bridge is presented in Figure 5.3.6-3. It consists of six standard California 3 ft - 6 in. PC I-girders (CA I42) with a 7 in. CIP composite deck and two Type 736 concrete barriers (Note: new standards require Type 836 barriers).

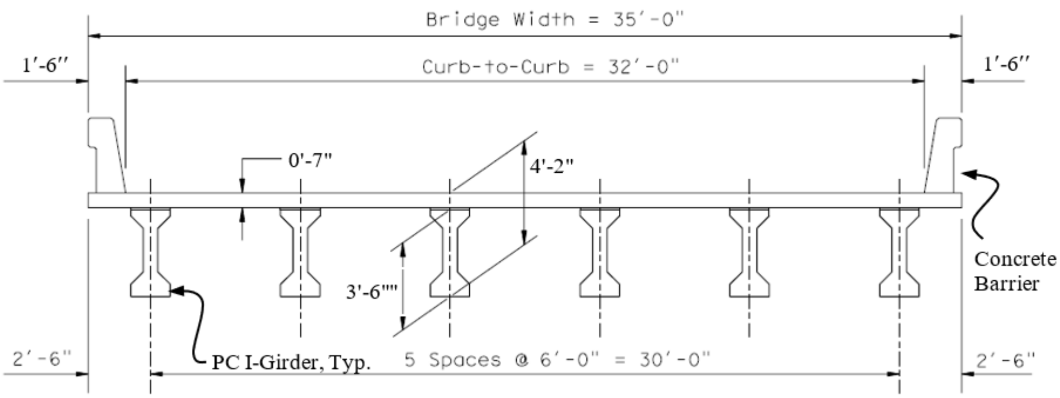


Figure 5.3.6-3 Typical Bridge Cross Section

5.3.6.3 Establish Loading Sequence

The loading sequence and corresponding stresses for a single-span PC girder are normally considered at three distinct stages, as summarized in Table 5.3.6-1. The table also indicates what section (non-composite versus composite) resists the applied loading.

Note: Per Caltrans practice, transportation (shipping and handling) is generally the responsibility of the Contractor and PC manufacturer.

Table 5.3.6-1 Typical Stages of Loading and Resisting Section for Single-Span PC Girder

Stage	Location	Construction Activity	Loads	Resisting Section
I	Casting Yard	Cast and Stress Girder (Transfer)	DC (Girder)	Girder (Non-composite)
IIA	On Site	Erect Girder, Cast Deck Slab	DC (Girder, Diaphragm, Slab), Construction Loads	Girder (Non-composite)
IIB	On Site	Construct Barrier Rails	DC (Girder, Diaphragm, Slab)	Girder (Non-composite)
			DC (Barrier Rails)	Girder and Deck (Composite)
III	Final	Open to Traffic	DC (Girder, Diaphragm, Slab) DC (Barrier Rails) DW (Future Wearing Surface)	Girder and Deck (Composite)
			DC (Girder, Diaphragm, Slab) DC (Barrier Rails) DW (Future Wearing Surface) LL (Vehicular Loading, HL-93 or P15)	Girder and Deck (Composite)

5.3.6.4 Select Materials

The following materials are selected for the bridge components. The concrete strengths for PC girders at transfer and at 28 days are assumed at this stage of design based on common practice in California. However, these values are subsequently verified during service limit state design:

- Concrete compressive strength and modulus of elasticity:
  - PC girder

    Concrete unit weight is assumed herein w_c = 0.15 kcf

    At transfer:

$f'_{ci} = 4.8$ ksi (80% of f'_c at 28 days)

$$E_{ci} = 120{,}000 K_1 w_c^{2.0} f'^{0.33}_{ci} \qquad \text{(AASHTO 5.4.2.4-1)}$$

$$= 120{,}000(1.0)(0.15)^{2.0}(4.8)^{0.33} = 4{,}531 \text{ ksi}$$

E_{ci} = modulus of elasticity of concrete at time of transfer

At 28 days:

f'_c = 6 ksi

$$E_c = 120{,}000(1.0)(0.15)^{2.0}(6)^{0.33} = 4{,}877 \text{ ksi}$$

- Cast-in-place deck slab

  Concrete unit weight is assumed herein w_c = 0.15 kcf

  f'_c = 4 ksi (Article 5.4.2.1 of CA)

  $$E_c = 120{,}000(1.0)(0.15)^{2.0}(4)^{0.33} = 4{,}266 \text{ ksi}$$

- Prestressing steel:

  0.6 in. diameter, seven-wire, low-relaxation strands,

  Area of each strand, A_{ps} = 0.217 in.^{2}

  Grade 270, nominal tensile strength,

  f_{pu} = 270 ksi (AASHTO Table 5.4.4.1-1)

  Yield strength, $f_{py} = 0.9 f_{pu}$ = 243 ksi (AASHTO Table 5.4.4.1-1)

  Initial jacking stress, $f_{pj} = 0.75 f_{pu}$ = 202.5 ksi (CA Table 5.9.2.2-1)

  Modulus of elasticity of prestressing steel,

  E_p = 28,500 ksi (AASHTO Article 5.4.4.2)

- Mild steel - A706 reinforcing steel:

  Nominal yield strength, f_y = 60 ksi

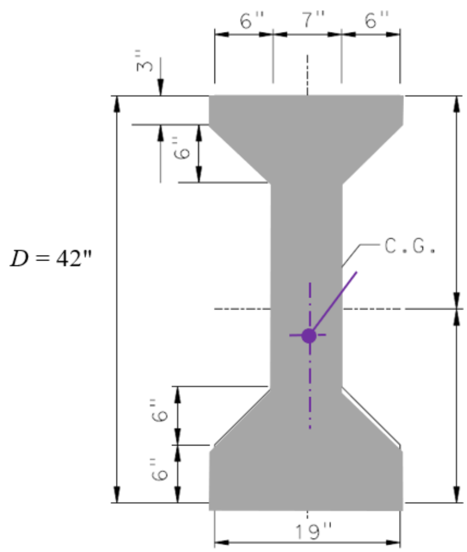
  Modulus of elasticity of steel, E_s = 29,000 ksi

5.3.6.5 Calculate Section Properties

In calculating section properties, gross sections are used for estimating the required prestress force (Section 5.3.6.8) and for estimating prestress losses using the AASHTO-CA BDS-8 Approximate Method (Section 5.3.6.9). However, girder flexural stresses are checked at the service limit state based on transformed section properties (Section 5.3.6.10).

5.3.6.5.1 Precast Section

Figure 5.3.6-4 shows the California Standard 3 ft 6 in. I girder (CA I42) and gross section properties of the girder. Section properties are obtained from BDM 5.3.



SECTION PROPERTIES
$A_g = 474$ in.^{2}
$I_{cg} = 95{,}400$ in.^{4}
$y_b = 20$ in.
$y_t = 22$ in.
$S_b = 4{,}770$ in.^{3}
$S_t = 4{,}336$ in.^{3}
$r = 14.2$ in.

Figure 5.3.6-4 Standard CA I42 Girder (BDM 5.3)

A_g = gross area of girder section (in.^{2})
I_g = gross moment of inertia of girder about centroidal axis (in.^{4})
y_b = distance from neutral axis to extreme bottom fiber of PC girder (in.)
y_t = distance from neutral axis to extreme top fiber of PC girder (in.)
S_b = section modulus for bottom extreme fiber of section (in.^{3})
S_t = section modulus for top extreme fiber of section (in.^{3})
r = radius of gyration (in.)

5.3.6.5.2 Effective Flange Width

Article 4.6.2.6 states that the effective flange width, b_{eff}, may be taken as the full flange width if $L/S \geq 2$, where S is spacing of girders or webs (ft); and L is individual span length (ft).

For this example,

$$\frac{L}{S} = \frac{70}{6} = 11.67 > 2$$

Therefore, the effective flange width b_{eff} = S = 72 in.

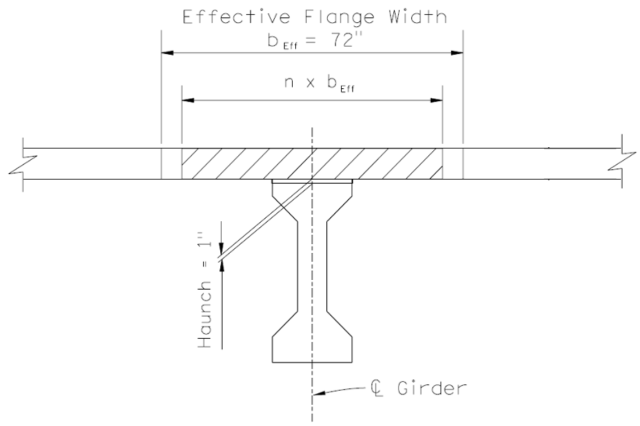


Figure 5.3.6-5 Effective Flange Width

5.3.6.5.3 Composite Section

To compute properties of the composite section, the CIP deck slab and haunch concrete (same material as deck) are transformed to the higher strength girder concrete using the modular ratio, n.

$$n = \frac{E_B}{E_D} \qquad \text{(AASHTO 4.6.2.2.1-2)}$$

where:

n = modular ratio between girder and deck

E_B = modulus of elasticity of girder material (ksi)

E_D = modulus of elasticity of deck material (ksi)

Using AASHTO Eq.4.6.2.2.1-2:

$$n = \frac{E_B}{E_D} = \frac{4,877}{4,266} = 1.14$$

Transformed flange width $= \dfrac{72}{n} = \dfrac{72}{1.14} = 63.2$ in.

Transformed deck area = 63.2(7) = 442.4 in.^{2}

Transformed haunch width $= \dfrac{19}{n} = \dfrac{19}{1.14} = 16.7$ in.

Transformed haunch area = 16.7(1) = 16.7 in.^{2}

Table 5.3.6-2 Section Properties - Gross Composite Section

Section	A_i (in.^{2})	y_i (in.)	$A_i(y_i)$ (in.^{3})	I_o (in.^{4})	$A_i(Y-y_i)^2$ (in.^{4})
Deck	442.4	46.5	20,571	1,807	80,627
Haunch	16.7	42.5	710	1	1,507
Girder	474	20	9,480	95,400	80,106
Total	933.1	-	30,761	97,208	162,241

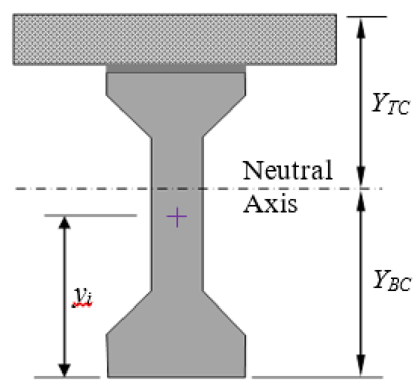
$A_c = 933.1 \text{ in.}^2$

$$Y_{BC} = \frac{\sum A_i y_i}{\sum y_i} = \frac{30{,}761}{933.1} = 33 \text{ in.}$$

$Y_{TC} = 50 - 33 = 17 \text{ in.}$

$$I_c = 97{,}208 + 162{,}241 = 259{,}449 \text{ in.}^4$$

$$S_{BC} = \frac{I_c}{Y_{BC}} = \frac{259{,}323}{33} = 7{,}862 \text{ in.}^3$$



where:

y_i = distance from centroid of section *i* to centroid of composite section

A_c = concrete area of composite section

Y_{TC} = distance from centroid of composite section to extreme top fiber of composite section

I_c = moment of inertia of composite section

S_{BC} = section modulus of the composite section for extreme bottom fiber of PC girder

5.3.6.6 Determine Loads

5.3.6.6.1 Dead Load

PC Girder:

$$w_g = \frac{474}{144}(0.15) = 0.494 \text{ klf}$$

Slab (before reaching design strength):

$$w_s = \frac{504}{144}(0.15) = 0.525 \text{ klf}$$

Haunch:

$$w_h = \frac{19}{144}(0.15) = 0.020 \text{ klf}$$

Dead loads on composite section:

Type 732 barrier rail on both sides of deck (concrete area = 444 in^2): $w_{br} = \frac{444}{144}(0.15) = 0.463$ klf/barrier

Dead load of wearing surfaces and utilities - *DW* (Article 3.3.2, AASHTO, 2017)

3 in. polyester concrete overlay = 0.035 ksf (CA Article 3.5.1)

5.3.6.6.2 Live Load

At the Service Limit State, AASHTO-CA BDS-8 requires design for the HL-93 vehicular live load. At the Strength Limit State, AASHTO-CA BDS-8 requires design for both HL-93 vehicular live load and the California P15 permit truck.

- HL-93 vehicular live load consists of these combinations:
  - Design truck or design tandem (Article 3.6.1.2.1)
  - Design lane load of 0.64 klf without dynamic load allowance (IM) (Article 3.6.1.2.4)
- California P15 permit truck: The P15 vehicular live load is the California P15 Permit Design Truck defined in Article 3.6.1.8 of California Amendments (Caltrans, 2019a).

5.3.6.7 Perform Structural Analysis

5.3.6.7.1 Dead Load Distribution Factor

According to Article 4.6.2.2.1 , permanent dead loads (including concrete barriers and wearing surface) may be distributed uniformly among all girders provided all of the following conditions are met:

- Width of deck is constant. (OK)
- Number of girders, N_b, is not less than four; i.e., $N_b = 6$ (OK)

- Girders are parallel and have approximately the same stiffness. (OK)
- Roadway part of the overhang, d_e, does not exceed 3 ft d_e is taken as the distance from the exterior web of exterior girder to interior edge of curb:
- d_e = 2.5 - 1.5 - 0.5(7/12) = 0.71 ft ≤ 3 ft (OK)
- Bridge is on a tangent line and curvature in plan is zero. (OK)
- Cross-section is consistent with one of the cross-sections shown in AASHTO Table 4.6.2.2.1-1 (AASHTO, 2017). The superstructure is type (*k*). (OK)

Because the design example satisfies the criteria, the concrete barrier and wearing surface loads can be evenly distributed among the six girders based on the dead load distribution factor (DF_{DL}), which is determined as:

$$DF_{DL} = \frac{\text{Tributary Width}}{\text{Bridge Width}} = \frac{6}{35} = 0.171$$

Using the DF_{DL}:

Barrier, w_{br} = $DC3$ = (0.463)(2)(0.171) = 0.159 klf/girder

DW = dead load of future wearing surface, 0.035 ksf

DW = (0.035)(32)(0.171) = 0.192 klf/girder

5.3.6.7.2 Unfactored Shear Force and Bending Moment due to DC and DW

Dead load shear and moment can be obtained from structural analysis software or can be calculated as follows (for simply-supported, single-span bridges):

Shear at x, $V_x = w\,(0.5L - x)$

Moment at x, $M_x = 0.5wx\,(L - x)$

where:

w = uniform dead load, klf

x = distance from left end of girder (ft)

L = span length = 70 ft

Table 5.3.6-3 Unfactored Shear Force and Bending Moment due to *DC* and *DW*

Location		Girder Weight (*DC*1)		Slab, Haunch Wt. (*DC*2)		Barrier Weight (*DC*3)		Future Wearing Surface (*DW*)	
Dist/Span (*X*/*L*)	Location (ft)	Shear (kip)	Moment (kip-ft)	Shear (kip)	Moment (kip-ft)	Shear (kip)	Moment (kip-ft)	Shear (kip)	Moment (kip-ft)
0 *L*	0	17.3	0	19.1	0	5.6	0	6.7	0
0.05*L**	3.5	15.6	57.5	17.2	63.4	5	18.5	6	22.3
0.1*L*	7	13.8	108.9	15.3	120.1	4.4	35	5.4	42.3
0.2*L*	14	10.4	193.6	11.4	213.6	3.3	62.2	4	75.3
0.3*L*	21	6.9	254	7.6	280.3	2.2	81.6	2.7	98.8
0.4*L*	28	3.5	290.3	3.8	320.3	1.1	93.2	1.3	112.9
0.5*L*	35	0	302.4	0	333.7	0	97.4	0	117.6

*Critical shear section

5.3.6.7.3 Unfactored Shear Force and Bending Moment due to Live Loads

Live loads are applied to the bridge deck on one or more design lanes. Therefore, shear forces and bending moments are normally calculated on a per-lane basis. However, shear forces and moments must then be distributed to individual girders for girder design. AASHTO-CA BDS-8 permits governing values of shear force and moment envelopes to be distributed to individual girders using simplified distribution factor formulas, specified separately for moment and shear (Articles 4.6.2.2.2 and 4.6.2.2.3, respectively). As shown previously, the conditions of Article 4.6.2.2 are satisfied for this example bridge. Therefore, the simplified distribution factor formulas are applied to the interior girder design in the following sections.

5.3.6.7.3.1 Live Load Moment Distribution Factor, DFM (for Interior Girders)

The live load distribution factor for moment (DFM, lanes/girder), for an interior girder is governed by the larger value for one design lane versus two design lanes loaded, as shown below.

- One design lane loaded:

$$DFM = 0.060 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12L\left(t_s\right)^3}\right)^{0.1} \quad \text{(AASHTO Table 4.6.2.2.2b-1)}$$

Provided the following ranges are met:

$3.5 \leq S \leq 16$

S = girder spacing = 6 ft (OK)

$4.5 \leq t_s \leq 12$

t_s = thickness of concrete slab = 7 in. (OK)

$20 \leq L \leq 240$

L = span length = 70 ft (OK)

N_b = number of girders ≥ 4

N_b = 6 (OK)

$10{,}000 \leq K_g \leq 7{,}000{,}000$

Longitudinal stiffness parameter, K_g = 488,224 in.^{4} (OK)

See calculation below:

$K_g = n(I + Ae_g^2)$ (AASHTO 4.6.2.2.1-1)

$n = E_B / E_D = 1.14$ (AASHTO 4.6.2.2.1-2)

$I = I_{cg} = 95{,}400$ in.^{4}

$A = A_g = 474$ in.^{2}

e_g = distance between centers of gravity of girder and deck

= 46.5 – 20 = 26.5 in.

$K_g = 1.14\,[95{,}400 + 474\,(26.5)^2] = 488{,}224$ in.^{4}

$$DFM = 0.060 + \left(\frac{6}{14}\right)^{0.4}\left(\frac{6}{70.0}\right)^{0.3}\left(\frac{488{,}224}{12\,(70)\,(7)^3}\right)^{0.1}$$

$$= 0.06 + (0.713)(0.479)(0.836) = 0.346 \text{ lanes/girder}$$

- Two or more design lanes loaded:

$$DFM = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12L\,(t_s)^3}\right)^{0.1}$$ (AASHTO Table 4.6.2.2.2b-1)

$$DFM = 0.075 + \left(\frac{6}{9.5}\right)^{0.6}\left(\frac{6}{70}\right)^{0.2}\left(\frac{488{,}224}{12\,(70)\,(7)^3}\right)^{0.1}$$

$$= 0.075 + (0.759)(0.612)(0.836) = 0.463 \text{ lanes/girder}$$

Therefore, *DFM* for two or more lanes loaded is larger and thus controls. Use *DFM* = 0.463 lanes/girder

5.3.6.7.3.2 Live Load Shear Distribution Factor (DFV) for Interior Girders

- One design lane loaded:

$$DFV = 0.36 + \left(\frac{S}{25}\right) \quad \text{(AASHTO Table 4.6.2.2.3a-1)}$$

$$= 0.36 + 0.24 = 0.6 \text{ lanes/girder}$$

- Two or more design lanes loaded:

$$DFV = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^2$$

$$= 0.2 + 0.5 - 0.029 = 0.671 \text{ lanes/girder}$$

Therefore, *DFV* for two or more lanes loaded is larger and thus controls. Use *DFV* = 0.671 lanes/girder

Note: The dynamic load allowance factor (IM) is applied to the HL-93 design truck and P15 permit truck only, not to the HL-93 design lane load. Table 3.6.2.1-1 of California Amendments (Caltrans, 2019a) summarizes the values of IM for various components and load cases.

The live load moment and shear are commonly calculated at tenth points and can be obtained from common structure analysis programs. Spreadsheets can also be used for simple-span structures. In this example, structural analysis software was used to determine the live load moments. The results are tabulated in Table 5.3.6-4 for HL-93 loading and Table 5.3.6-5 for P15 loading, respectively. These tables list the envelope values for moment and shear per lane, as well as per girder (for design) using the distribution factors.

Table 5.3.6-4 Unfactored Live Load Moment and Shear Force Envelope Values due to HL-93 (*LL* + *IM*)

Location		Per Lane†		*DFM*	*DFV*	Per Girder	
	(ft)	Moment (kip-ft)	Shear (kip)	(Lane per Girder)	(Lane per Girder)	$M_{(LL+IM)}$ (kip-ft)	$V_{(LL+IM)}$ (kip)
0*L**	0	0	102.11	0.463	0.671	0	68.5
0.05*L***	3.5	348.5	97.9	0.463	0.671	199	65.7
0.1*L*	7	655.03	91.56	0.463	0.671	373.8	61.4
0.2*L*	14	1144.64	78.18	0.463	0.671	653.2	52.4
0.3*L*	21	1468.82	65.24	0.463	0.671	838.2	43.8
0.4*L*	28	1657.38	52.75	0.463	0.671	945.8	35.4
0.5*L*	35	1695.40	-40.87	0.463	0.671	967.5	-27.4

*L = Span Length
** Critical section for shear
†These values were obtained from CT Bridge (Include IM = 33%)

Table 5.3.6-5 Unfactored Live Load Moment and Shear Force Envelope Values due to P15 Truck (*LL* + *IM*)

Location		Per Lane†		*DFM*	*DFV*	Per Girder	
	(ft)	Moment (kip-ft)	Shear (kip)	(Lane per Girder)	(Lane per Girder)	$M_{(LL+IM)}$ (kip-ft)	$V_{(LL+IM)}$ (kip)
0*L**	0	0	178.5	0.463	0.671	0	119.8
0.05*L***	3	532.4	152.3	0.463	0.671	304	102.2
0.1*L*	7	972	138.86	0.463	0.671	554.7	93.1
0.2*L*	14	1566	111.86	0.463	0.671	893.6	75
0.3*L*	21	2025	89.68	0.463	0.671	1,155.6	60.1
0.4*L*	28	2349	69.43	0.463	0.671	1,340.5	46.6
0.5*L*	35	2328.75	-50.14	0.463	0.671	1,328.9	-33.6

*L = Span Length
** Critical section for shear
†These values were obtained from CT Bridge (Include IM = 25%)

5.3.6.8 Estimate Prestressing Force and Area of Strands

The minimum jacking force, P_j and associated area of prestressing strands, A_{ps}, can be reasonably estimated based on satisfying the two tensile stress limits at the bottom fiber of the PC girder at the Service III limit state:

- Case A) No tension under permanent loads
- Case B) Tension limited to prevent cracking under total dead and live loads

It should be noted that, for Service III, only the HL-93 vehicular live load applies. P15 applies to Strength II but not Service III. The critical location for bending moment is normally midspan. However, other locations such as 0.4*L* (P15 truck) and harp points can govern and must be checked as well. Gross section properties are used.

Calculations for these two critical cases are provided below.

Note: Compression is taken as positive (+) and tension as negative (-).

- Case A: No tension is allowed for components with bonded prestressing tendons or reinforcement, subjected to permanent loads (*DC*, *DW*) only. Set the stress at the bottom fiber equal to zero and solve for the required effective prestress force (at service, i.e., after losses), *P*, to achieve no tension.

$$\frac{P}{A_g}+\frac{Pe_c}{S_b}-\left(\frac{M_{DC1}+M_{DC2}}{S_b}+\frac{M_{DC3}+M_{DW}}{S_{BC}}\right)=0$$

Rearranging the equation:

$$P=\frac{\left(\dfrac{M_{DC1}+M_{DC2}}{S_b}+\dfrac{M_{DC3}+M_{DW}}{S_{BC}}\right)}{\dfrac{1}{A_g}+\dfrac{e_c}{S_b}}$$

As shown in Table 5.3.6-3 (*DC* and *DW*) and Table 5.3.6-4 (HL-93 vehicular live load), the maximum moment due to dead load and live load occurs at midspan. Moments on a per girder basis are used for girder design.

M_{DC1} = unfactored moment due to girder self-weight

= 302.4 kip-ft

M_{DC2} = unfactored moment due to slab and haunch weight

= 333.7 kip-ft

M_{DC3} = unfactored moment due to barrier weight

= 97.4 kip-ft

M_{DW} = unfactored moment due to future wearing surface

= 117.6 kip-ft

S_{BC} = section modulus for the bottom extreme fiber of the composite section
= 7,862 in.^{3}

To solve for P, the required effective prestressing force, an estimate of the eccentricity of the noncomposite girder, e_c, is needed. To determine e_c, the centroid of the prestressing force at midspan can be reasonably estimated to be 4 in. from the bottom of the girder.

Thus, the eccentricity of prestressing force at midspan based on the noncomposite section is taken as:

e_c = 20 – 4 = 16 in.

$$P = \frac{\left(\frac{(302.4+333.7)(12)}{4{,}770} + \frac{(97.4+117.6)(12)}{7{,}862}\right)}{\frac{1}{474} + \frac{16}{4{,}770}}$$

Required effective prestressing force, P = 353.9 kip

- Case B: Allowable tension for components subjected to the Service III limit state (DC, DW, (0.8) HL-93), subjected to not worse than moderate corrosion conditions, and located in Non-Freeze/Thaw Areas $= -0.19\sqrt{f'_c}$.

$$\frac{P}{A_g} + \frac{Pe_c}{S_b} - \left(\frac{M_{DC1} + M_{DC2}.}{S_b} + \frac{M_{DC3} + M_{DW} + 0.8(M_{HL93})}{S_{BC}}\right) = -0.19\sqrt{f'_c}$$

where:

M_{HL93} = moment due to HL-93 loading at midspan = 967.5 kip-ft (Table 5.3.6-4)

$$P = \frac{\left(\frac{M_{DC1} + M_{DC2}.}{S_b} + \frac{M_{DC3} + M_{DW} + 0.8(M_{HL93})}{S_{BC}}\right) - (0.19)\sqrt{f'_c}}{\frac{1}{A_g} + \frac{e_c}{S_b}}$$

$$P = \frac{\left(\frac{(302.4+333.7)(12)}{4{,}770}\right) + \left(\frac{[97.4+117.6+0.8(967.5)](12)}{7{,}862}\right) - 0.19\sqrt{6}}{\frac{1}{474} + \frac{16}{4{,}770}}$$

Required effective prestressing force, P = 484 kip

The minimum required effective prestressing force, P, at service level for an interior girder is the larger value from Case A and Case B. Therefore, $P = P_f$ = 484 kip/girder.

To determine the minimum required jacking force, an estimate of prestress losses is needed. Thus, assuming total (immediate and long-term) prestress losses of 25% (of the jacking force), the required jacking force (i.e., just before transfer, ignoring minor losses from jacking to de-tensioning) is:

Minimum Jacking Force, $P_j = \dfrac{484}{0.75} = 645.3 \text{ kip}$

The required area of prestressing strands, A_{ps}, jacked to 0.75 f_{pu} is:

Required $A_{ps} = \dfrac{645.3}{0.75(270)} = 3.19 \text{ in.}^2$

Number of 0.6 in. diameter strands required

$$= \frac{3.19}{0.217} = 14.7 \text{ strands}$$

14.7 is rounded to 16, an even number provided for symmetry (about a vertical line through the centroid) to produce a uniform stress distribution in the member.

Therefore, use sixteen 0.6 in. diameter low relaxation Grade 270 strands. The actual area of strands is thus:

A_{ps} = 16 (0.217) = 3.42 in.^{2}

Total prestressing force at jacking, P_j = 0.75(270)(3.42) = 703 kip

It is a common practice in Caltrans to provide contractors with the prestressing force and centroid of prestressing path on contract plans, instead of actual strand patterns. This gives the contractors flexibility in choosing the location and number of strands, based on the setup of their casting bed. However, designers are encouraged to layout an actual strand pattern. This helps ensure the design is constructible and avoids the possible use of too many strands in one girder.

The strand pattern is shown in Figure 5.3.6-6: six strands at 2.5 in., eight at 4.5 in. and two at 6.5 in.

The *CGS* from the bottom of the girder is:

$$CGS = \frac{6(2.5) + 8(4.5) + 2(6.5)}{16}$$

= 4 in. from bottom of girder.

The actual eccentricity, e_c, at midspan for the girder = 20 – 4 = 16 in., matching the assumption used in estimating the prestressing force. Normally, the actual value will vary from the assumption and should be used in subsequent design calculations.

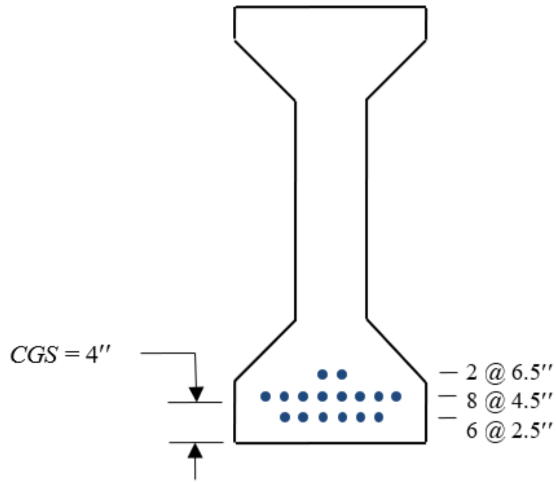


Figure 5.3.6-6 Strand Pattern in PC Girder at Midspan Section

5.3.6.9 Estimate Prestress Losses

Prestress losses were previously estimated in a very approximate way to determine area of strands. With a trial number of strands and layout now determined, prestress losses can be more accurately approximated.

Per AASHTO-CA BDS-8, , total prestress losses in prestressing strand stress are assumed to be the sum of immediate and long-term losses. Immediate losses for strands in a PC girder are due to elastic shortening. Long-term losses are primarily due to concrete creep and shrinkage as well as steel relaxation.

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \qquad \text{(AASHTO 5.9.3.1-1)}$$

where:

Δf_{pES} = change in stress due to elastic shortening loss (ksi)

Δf_{pLT} = losses due to long-term shrinkage and creep of concrete and relaxation of prestressing steel (ksi)

Δf_{pT} = total change in stress due to losses (ksi)

5.3.6.9.1 Elastic Shortening

Immediate elastic shortening losses are easily determined for PC girders using a closed form solution based on AASHTO Eq. C5.9.3.2.3a-1:

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt}\left(I_g + e_m^2 A_g\right) - e_m M_g A_g}{A_{ps}\left(I_g + e_m^2 A_g\right) + \dfrac{A_g I_g E_{ci}}{E_p}}$$

where:

A_{ps} = area of prestressing steel = 3.472 in.^{2}

A_g = gross area of girder section = 474 in.^{2}

f_{pbt} = stress in prestressing steel immediately prior to transfer

= 0.75(270) = 202.5 ksi, ignoring minor relaxation losses after jacking

E_{ci} = 4,531 ksi

E_p = 28,500 ksi

e_m = eccentricity at midspan = 16 in.

I_g = moment of inertia of gross section = 95,400 in.^{4}

M_g = midspan moment due to self-weight of girder

= M_{DC1} = 302.4 k-ft (12 in./ft) = 3,629 k-in.

$$\Delta f_{pES} = \frac{3.472(202.5)\left[95{,}400 + 16^2(474)\right] - 16(3{,}629)(474)}{3.472[\left(95{,}400 + 16^2(474)\right] + \dfrac{474(95{,}400)(4{,}531)}{28{,}500}}$$

$$\Delta f_{pES} = 15.72 \text{ ksi}$$

The initial prestressing stress immediately after transfer = 202.5 – 15.72 = 186.8 ksi.

Article C5.9.3.2.3a notes that when transformed section properties are used in calculating concrete stresses, the effects of losses and gains due to elastic deformation are implicitly accounted for, therefore, Δf_{pES} should not be used to reduce the stress in the prestressing strands (and force) for concrete stress calculations at transfer and service level.

5.3.6.9.2 Long Term Losses – Approximate Method

AASHTO-CA BDS-8 provides two methods to estimate the time-dependent prestress losses: Approximate Method (Article 5.9.3.3) and Refined Method (Article 5.9.3.4). This example uses the Approximate Method to estimate long-term, time-dependent prestress losses, based on gross section properties.

Per Article 5.9.3.3, the approximate method is applicable to standard precast, pretensioned members subject to normal loading and environmental conditions, where:

- Members are made from normal-weight concrete (OK)
- Concrete is either steam- or moist-cured (OK)
- Prestressing strands use low relaxation properties (OK)
- Average exposure conditions and temperatures characterize the site (OK)

Because the girder in this example satisfies all of the criteria, the Approximate Method can be used.

Long-term prestress losses due to creep and shrinkage of concrete and relaxation of steel are estimated using the following formula, in which the three terms correspond to creep, shrinkage, and relaxation, respectively:

$$\Delta f_{pLT} = 10\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12\gamma_h\gamma_{st} + \Delta f_{pR} \qquad \text{(AASHTO 5.9.3.3-1)}$$

where:

f_{pi} = prestressing steel stress immediately prior to transfer (ksi)

H = the average annual ambient relative humidity (%)

γ_h = correction factor for relative humidity of ambient air

= $1.7 - 0.01H$

γ_{st} = correction factor for specified concrete strength time at of prestress transfer to concrete member

= $5/(1 + f'_{ci})$

Δf_{pR} = an estimation of relaxation loss taken as 2.4 ksi for low relaxation strand

For this calculation:

f_{pi} = 202.5 ksi

H = Average annual ambient relative humidity = 70%

γ_h = $1.7 - 0.01H = 1.7 - 0.01\,(70) = 1$ (AASHTO 5.9.3.3-2)

$$\gamma_{st} = \frac{5}{\left(1 + f'_{ci}\right)} = \frac{5}{\left(1 + 4.8\right)} = 0.862 \qquad \text{(AASHTO 5.9.3.3-3)}$$

Δf_{pR} = 2.4 ksi for low relaxation strands

$$\Delta f_{pLT} = (10)\frac{(202.5)(3.472)}{474}(1)(0.862) + (12)(1)(0.862) + 2.4$$

$$= 12.8 + 10.3 + 2.4 = 25.5 \text{ ksi}$$

Total prestress losses:

Δf_{pT} = 15.7 + 25.5 = 41.2 ksi

$$\Delta f_{pT} = \frac{41.2}{202.5}(100\%) = 20\%$$

The strand stress increases with applied load at service, which is calculated as:

$$\text{Elastic gains} = n\left(\frac{M_{DC2}}{Sb} + \frac{M_{DC3} + M_{DW} + M_{LL}}{Sbc}\right)$$

where:

M_{DC2} = 333.7 kip-ft (Table 5.3.6-3)

M_{DC3} = 97.4 kip-ft (Table 5.3.6-3)

M_{DW} = 117.6 kip-ft (Table 5.3.6-3)

M_{LL} = 967.5 kip-ft (Table 5.3.6-4)

$$n = \frac{E_{ps}}{E_c} = \frac{28500}{4877} = 5.84$$

$$\text{Elastic gains} = 5.84\left(\frac{(333.7)(12)}{4770} + \frac{(12)(97.4 + 117.6 + 967.5)}{7862}\right) = 15.5 \text{ ksi}$$

fpe = effective stress in prestressing strands using gross non-transformed section properties (service limit state)

= 202.5 – 41.2 + 15.5 = 176.8 ksi

Check prestressing stress limit at service limit state:

$0.8\ f_{py} \geq f_{pe}$ (Table 5.9.2.2-1, Caltrans 2019)

0.8 (270) (0.9) = 194.4 ksi > 176.8 ksi, (OK)

For gross non-transformed sections, the effective prestressing force after all losses, P_f = 3.472 (176.8) = 613.8 kip

Regarding transformed sections, note that with transformed sections used in refined subsequent sections of this design example, the prestressing force to be used in concrete stress calculations at transfer is the jacking force, P_j, and the prestressing force to be used in concrete stress calculations at service level is the final prestressing force, P_f, based on long-term losses only:

Effective prestress used with transformed sections:

Check prestressing stress limit at service limit state:

$0.8\ f_{py} \geq f_{pe}$ (CA Amend Table 5.9.2.2-1)

0.8 (270) (0.9) = 194.4 ksi

$f_{pe} = 0.75\ f_{pu} - \Delta f_{pLT}$ = 0.75 (270) – 25.5 = 177 ksi <194.4 ksi (OK)

Note: When determining the stress using transformed section properties, all the elastic losses and gains are implicitly accounted for. Only total time-dependent losses need to be accounted for.

Therefore, $P_f = f_{pe}\ (A_{ps})$ = 177(3.472) = 614.5 kip

5.3.6.10 Design for Service Limit State

Design for the Service Limit State addresses the suitability of the previously estimated strand force and profile based on Stages I, IIA, and III. Concrete stresses are checked at transfer, which may lead to design modifications such as adjusting the strand profile or initial concrete compressive strength f′ci. The most critical check of stresses at the Service Limit State is normally the check of the tensile stress at the bottom of the girder to prevent possible cracking at Service III (HL-93 vehicular live load).

5.3.6.10.1 Calculate Transformed Section Properties

The use of transformed concrete section properties generally leads to more accurate calculations than with the use of gross section properties. As this is recognized by Article C5.9.3.2.3a, calculations use transformed section properties.

Three sets of transformed section properties are needed for the service limit state design. These include transformed section properties of the noncomposite girder at transfer (Stage I), erection, and deck casting (Stage IIA, before deck hardening), as well as the composite section of the girder and deck at service (Stage III). The section property calculations for these stages are presented below. A minor difference in final and initial transformed noncomposite properties results from the use of E_c versus E_{ci}. However, the difference between the composite and noncomposite final properties is significant due to the additional deck area.

Table 5.3.6-6 Transformed Section Properties: Girders and Strands (Initial, Transfer)

Section	A_i (in.^{2})	y_i (in.)	$A_i(y_i)$ (in.^{3})	I_i (in.^{4})	$A_i(Y-y_i)^2$ (in.^{4})
Girder	474	20	9,480	95,400	171
Strands	18.4‡	4	73.6	≈ 0	4,364
Total	492.4	--	9,553.6	95,400	4,535

$$n - 1 = \frac{E_{ps}}{E_{ci}} - 1 = \frac{28{,}500}{4{,}531} - 1 = 5.29$$

Total $A_c = 492.4$ in.^{2}

$$Y_{Btf} = \frac{9{,}553.6}{492.4} = 19.40 \text{ in.}$$

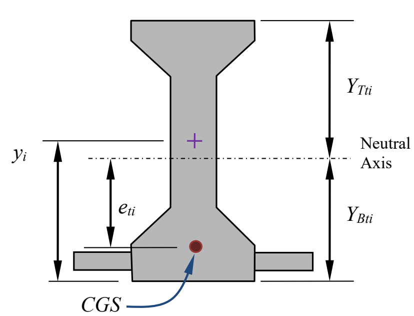
$Y_{Tti} = 42 - 19.40 = 22.6$ in.

$I_{ti} = 95{,}400 + 4{,}535 = 99{,}935$ in.^{4}

$$S_{Bti} = \frac{I_{ti}}{Y_{Bti}} = \frac{99{,}935}{19.40} = 5{,}151 \text{ in.}^3$$

$$S_{Tti} = \frac{I_{ti}}{Y_{Tti}} = \frac{99{,}935}{22.60} = 4{,}422 \text{ in.}^3$$

$e_{ti} = 19.4 - 4 = 15.4$ in.



‡Strands are transformed using (n – 1)

Table 5.3.6-7 Transformed Section Properties: Girders and Strands (Final)

Section	A_i (in.^{2})	y_i (in.)	$A_i(y_i)$ (in.^{3})	I_i (in.^{4})	$A_i(Y\text{-}y_i)^2$ (in.^{4})
Girder	474	20	9,480	95,400	143
Strands	16.8‡	4	67.2	≈ 0	4,010
Total	490.8	--	9,547.2	95,400	4,153

$$n - 1 = \frac{E_{ps}}{E_c} - 1 = \frac{28{,}500}{4{,}877} - 1 = 4.84$$

Total $A_c = 490.8$ in.^{2}

$$Y_{Btf} = \frac{9{,}547.2}{490.8} = 19.45 \text{ in.}$$

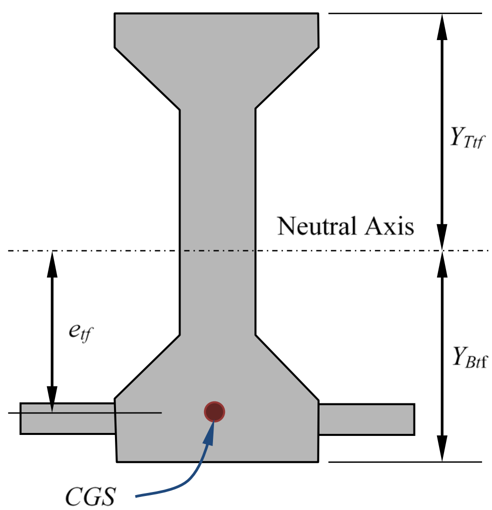
$Y_{Ttf} = 42 - 19.45 = 22.55$ in.

$I_{tf} = 95{,}4000 + 4{,}153$
$= 99{,}553$ in.^{4}

$$S_{Btf} = \frac{I_{tf}}{Y_{Btf}} = \frac{99{,}553}{19.45} = 5{,}117 \text{ in.}^3$$

$$S_{Ttf} = \frac{I_{tf}}{Y_{Ttf}} = \frac{99{,}553}{22.55} = 4{,}414 \text{ in.}^3$$

$e_t = 19.45 - 4 = 15.45$ in.



‡Strands are transformed using (n – 1)

Table 5.3.6-8 Transformed Section Properties: Composite Girder and Deck (Final)

Section	A_i (in.^{2})	y_i (in.)	$A_i(y_i)$ (in.^{3})	I_i (in.^{4})	$A_i(Y-y_i)^2$ (in.^{4})
Girder	474	20	9,480	95,400	72,882
Strands	16.8	4	67.2	≈ 0	13,550
Deck	438.5**	46.5	20,390.3	1,595	87,178
Haunch	16.5**	42.5	701.3	1	1,683
Total	945.8	-	30,638.8	96,996	175,293

$$n = \frac{E_{deck}}{E_{girder}} = \frac{4{,}266}{4{,}877} = 0.87$$

$$A_c = 945.8 \text{ in.}^2$$

$$Y_{BCt} = \frac{30{,}638.8}{945.8} = 32.4 \text{ in.}$$

$$Y_{TGCt} = 42 - 32.4 = 9.6 \text{ in}$$

$$Y_{TCt} = 50 - 32.4 = 17.6 \text{ in}$$

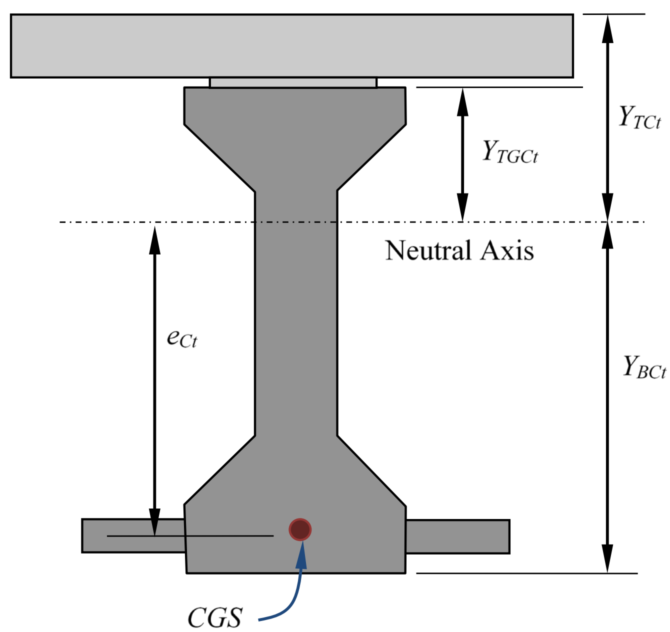
$$I_{Ct} = 96{,}996 + 175{,}293 = 272{,}289 \text{ in.}^4$$

$$S_{BCt} = \frac{I_{Ct}}{Y_{BCt}} = \frac{272{,}289}{32.4} = 8{,}404 \text{ in.}^3$$

$$S_{TGCt} = \frac{I_{Ct}}{Y_{TGCt}} = \frac{272{,}289}{9.6} = 28{,}363 \text{ in.}^3$$

$$S_{TDCt} = \frac{I_{Ct}}{Y_{DCt}} = \frac{272{,}289}{17.6(0.87)} = 17{,}783 \text{ in.}^3$$

$$e_{Ct} = 32.4 - 4 = 28.4 \text{ in.}$$



**Deck and haunch are transformed using (n = 0.87)

5.3.6.10.2 Check Concrete Stresses at Transfer Condition

The check of concrete stresses at transfer investigates the suitability of both the prestressing force and the strand profile for the assumed section. Commonly, strands must be harped or debonded to produce an efficient design that does not overstress the section. In addition, the initial concrete compressive strength, f'_{ci} may be modified.

- Concrete stress limits:
  - ✓ Compressive stress limit:

    Stress limit = 0.65 f'_{ci} = 0.65 (4.8) = 3.120 ksi (AASHTO 5.9.2.3.1)
  - ✓ Tensile stress limit: (AASHTO Table 5.9.2.3.1b-1)
    - In area other than precompressed tensile zone without bonded auxiliary reinforcement:

      Stress Limit = $0.0948\sqrt{f'_c} \leq 0.200$ ksi (λ=1, normal weight concrete)

      Stress Limit = $0.0948\sqrt{4.8} = 0.208$ ksi

      Since 0.208 ksi is larger than 0.200 ksi, 0.200 ksi is taken as the limit.
    - In areas with bonded auxiliary reinforcement sufficient to resist the tensile force

      Stress Limit = $0.24\sqrt{4.8} = 0.526$ ksi

Per Article C5.9.3.2.3a, when checking concrete stresses using transformed section properties, the effects of losses and gains due to elastic deformations are implicitly accounted for. Therefore, the elastic shortening loss, Δf_{pES}, should not be subtracted from the strand stress in calculating the prestressing force at transfer (taken as P_j because relaxation losses between jacking and transfer are ignored).

Check concrete stresses at transfer length section: (straight strands)

Transfer length = 60(d_b) = 60(0.6) = 36 in. = 3 ft (AASHTO Figure C5.9.4.3.2-1)

d_b = nominal strand diameter (in.)

P_j = 703 kip

A_{ti} = 492.4 in.^{2}

where:

A_{ti} = Gross area of girder concrete at time of force transfer

Eccentricity at 3 ft with *CGS* strands = 4 in. from bottom

e_{ti} = 15.4 in.

S_{Bti} = 5,151 in.^{3}

S_{Tti} = 4,422 in.^{3}

M_{DC1} = moment at transfer length section due to girder weight, based on total girder length

= 0.5(0.494)(3)(71 - 3) = 49.8 kip-ft = 598 kip-in.

Calculate concrete stress at top of girder at transfer length:

$$f_{top} = \frac{P_j}{A_{ti}} - \frac{P_j e_{ti}}{S_{Tti}} + \frac{M_{DC1}}{S_{Tti}} = \frac{703}{492.4} - \frac{703(15.4)}{4,422} + \frac{598}{4,422} = -0.885 \text{ksi (Tensile)}$$

Because the tensile stress exceeds the upper limit of the allowable tensile stress (0.526 ksi, assuming bonded reinforcement), the effect of prestressing must be reduced. Either harping or debonding a portion of the strands near the end of the girders can accomplish this.

For this example, two point harping is selected. Harp points are usually located between 0.33*L* and 0.4*L* from girder ends. 0.4*L* is chosen as the harp point. The calculations below investigate stressses along the member by the use of four strands harped with a profile at the girder ends as follows: two strands at 28.5 in. from bottom and two strands 30.5 in. from bottom. A suitable configuation of harping is easily arrived at by iteration using spreadsheets or commercially available software. From midspan to the harped point (0.4*L*), *CGS* is 4 in. from the girder bottom, and at the girder ends, the *CGS* is 10 in. from the bottom. The strand patterns at the harp points and girder ends are shown in Figure 5.3.6-7. Hold-down forces and harp angle are normally calculated and checked against limits by local precast producers.

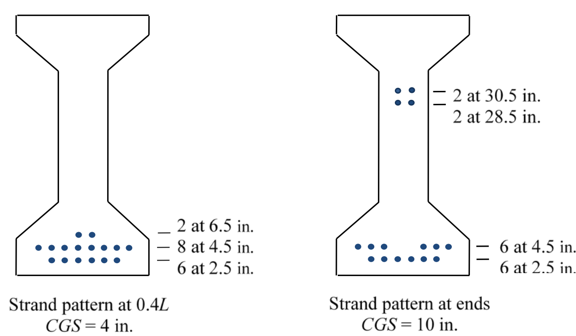


Figure 5.3.6-7 Strand Patterns at Midspan and Ends of Girder

- Check concrete stresses at transfer length section (harped strands):

  The eccentricity at transfer length (3 ft) with harped strands:

$$e_{ti} = \frac{28-3}{28}(10.0-4.0) + 4.0 = 9.4 \text{ in.}$$

$$f_{top} = \frac{P_j}{A_{ti}} - \frac{P_j e_{ti}}{S_{Tti}} + \frac{M_{DC1}}{S_{Tti}} = \frac{703}{492.4} - \frac{703(9.4)}{4,422} + \frac{598}{4,422}$$

$= 0.067 \text{ ksi (Compressive)} < 2.880 \text{ ksi} \quad \text{(OK)}$

Note that the transformed section properties at midspan are used in the calculations above. It is not necessary to check the stresses at transfer length using transformed properties at transfer length since the stress level is very low.

- Check concrete stresses at harp points:

Harped point location = 0.4*L* =0.4(70) = 28 ft

P_j = 703 kip

A_{ti} = 492.4 in.^{2}

Eccentricity at 0.4*L*, e_{ti} = 15.4 in.

S_{Bti} = 5,151 in.^{3}

S_{Tti} = 4,422 in.^{3}

M_{DC1} = moment at 0.4*L* due to girder weight, based on total girder length

M_{DC1} = 0.5 (0.494) (28) (70 – 28) = 290.3 kip – ft = 3,484 kip – in. (Table 5.3.6-3)

Calculate concrete stress at top of girder at transfer at 0.4*L*:

$$f_{top} = \frac{P_j}{A_{ti}} - \frac{P_j e_{ti}}{S_{Tti}} + \frac{M_{DC1}}{S_{Tti}} = \frac{703}{492.4} - \frac{703(15.4)}{4,422} + \frac{3,484}{4,422} = -0.233 \text{ ksi (Tensile)}$$

Since the tensile stress at top of girder exceeded the limit of 0.200 ksi, auxiliary (mild) reinforcement must be provided at the tensile face (top) to resist the total tensile force.

- Determine auxiliary reinforcement:

To find the neutral axis of the section, it is necessary to determine:

$$f_{bot} = \frac{P_j}{A_{ti}} + \frac{P_j e_{ti}}{S_{Bti}} - \left(\frac{M_{DC1}}{S_{Bti}}\right) = \frac{703}{492.4} + \frac{703(9.4)}{5,151} - \left(\frac{3,484}{5,151}\right)$$

$$= 2.034 \text{ ksi (Compressive)} < 2.88 \text{ ksi} \quad \text{(OK)}$$

Locate neutral axis, *x*, from similar triangles.

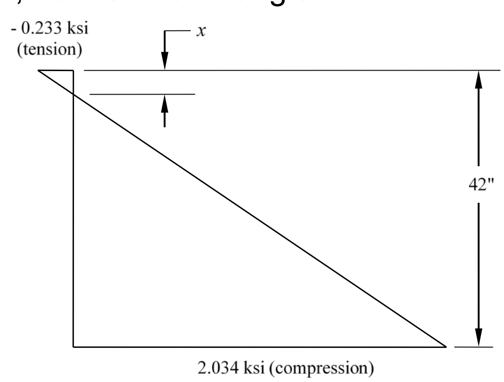


Figure 5.3.6-8 Concrete Stress Distribution

$$x = \frac{42(0.233)}{0.233 + 2.034} = 4.32 \text{ in.}$$

The neutral axis is below the 3 in. rectangular section of the top flange. The value 4.32 in. can be conservatively used to calculate the total tension force as the area of concrete times the average tensile stress.

Required tension capacity, F_T = 0.5 (0.227) (4.32) (19) = 9.32 kip

Using non-prestressed reinforcement (mild steel) at a working stress of 24 ksi

Area of mild steel required:

$$\frac{9.32}{24} = 0.39 \text{ in.}^2$$

Provide two #4 bars at top flange for entire length of girder,

$A_{s,provided}$ = 2 (0.2) = 0.4 in.^{2} > 0.39 in.^{2} (OK)

- Check concrete stresses at midspan:

  Midspan is not expected to govern over the harp points at transfer because of the beneficial effects of self-weight. However, the calculation is demonstrated herein. The values for P_j, e_{ti}, A_{ti}, S_{Bti}, and S_{Tti} at midspan are the same as at 0.4*L*.

  M_{DC1} = moment at midspan due to girder weight, based on total girder length = (0.494)(70)^2/8 = 302.4 kip-ft = 3,629 kip-in. (Table 5.3.6-3)

$$f_{top} = \frac{P_j}{A_{ti}} - \frac{P_j e_{ti}}{S_{Tti}} + \frac{M_{DC1}}{S_{Tti}} = \frac{703}{492.4} - \frac{703(15.4)}{4,422} + \frac{3,629}{4,422} = -0.200 \text{ ksi (Tensile)}$$

  This is within the tensile limit of 0.2 ksi (OK)

$$f_{bot} = \frac{P_j}{A_{ti}} - \frac{P_j e_{ti}}{S_{Bti}} + \frac{M_{DC1}}{S_{Bti}} = \frac{703}{492.4} - \frac{703(15.4)}{5,151} + \frac{3,629}{5,151}$$
$$= 2.825 \text{ ksi (Compressive)} < 2.880 \text{ ksi} \quad \text{(OK)}$$

5.3.6.10.3 Check Concrete Stresses at Service Condition

The check of concrete stresses at service level investigates the suitability of the section to resist service level loads. Of particular importance is the prevention of flexural cracking of the section at midspan for Service Level III due to the HL-93 Vehicular Live Load. In addition, per requirements of the California Amendments (Caltrans, 2019a), the section must not develop any tension under permanent loads (only).

- Effective prestressing force, P_f :

  Because transformed section properties are used, the effective prestressing force (P_f) acting on the section is calculated using the force at transfer, P_j, less long-term losses that were estimated using the Approximate Method:

$f_{pe} = 0.75\ f_{pu} - \Delta f_{pLT} = 0.75\ (270) - 25.5 = 177$ ksi

$P_f = f_{pe}\ (A_{ps}) = 177\ (3.472) = 614.5$ kip

- Concrete stress limits:

  - Compressive stress limits (AASHTO Table 5.9.2.3.2a-1)

    - Compressive stress limits due to unfactored permanent loads (including girder, slab and haunch, barrier, and future wearing surface) and prestressing force.

      Load combination: PS + Perm

      PC girder: $0.45\ f'_c = 0.45(6) = 2.700$ ksi

      CIP deck: $0.45\ f'_{c,deck} = 0.45(4.0) = 1.8$ ksi

    - Compression stress limit due to effective prestress, permanent, and transient loads (including all dead and live loads).

      Load combination: Service I = PS + Perm + $(LL + \text{IM})_{\text{HL-93}}$

      PC girder: $0.6\phi_w\ f'_c = 0.6(1.0)(6) = 3.600$ ksi

      CIP deck: $0.6\ \phi_w\ f'_{c,deck} = 0.6(1.0)(4.0) = 2.4$ ksi

    - Tensile stress limit (Table 5.9.2.3.2b-1, Caltrans 2019a)

  - For components with bonded prestressing tendons or reinforcement subjected to permanent loads only.

    Load combination: PS + Perm

    PC girder: 0 ksi (no tension allowed)

  - For components with bonded prestressing tendons or reinforcement.

    Load combination:

    Service III = PS + Perm + $0.8(LL+IM)_{HL\text{-}93}$

    PC girder: $-0.19\sqrt{f'_c} = -0.19\sqrt{6} = -0.465$ ksi

- Check concrete stresses at midspan:

  Bending moments at midspan are given in Table 5.3.6-9.

Table 5.3.6-9 Unfactored Bending Moments at 0.4L and Midspan (per Girder)

Location	*M_{DC1} (kip-ft)	*M_{DC2} (kip-ft)	*M_{DC3} (kip-ft)	*M_{DW} (kip-ft)	**$M_{(LL+IM)HL93}$ (kip-ft)
0.4*L*	290.3	320.3	93.2	112.9	945.8
0.5*L*	302.4	333.7	97.4	117.6	967.5

*** From Table 5.3.6-3**

**** From Table 5.3.6-4**

- Check compressive stresses at midspan:

  PC girders are checked for compressive stresses under the following two load combinations:

  ✓ Load combination PS + Perm

  - Stress at top of PC girder:

$$f_{tg} = \frac{P_f}{A_{tf}} - \frac{Pe_{tf}}{S_{Ttf}} + \frac{(M_{DC1} + M_{DC2})}{S_{Ttf}} + \frac{(M_{DC3} + M_{DW})}{S_{TGCt}}$$
$$= \frac{614.5}{490.8} - \frac{614.5(15.45)}{4,414} + \frac{(302.4 + 333.7)(12)}{4,414} + \frac{(97.4 + 117.6)(12)}{28,363}$$
$$= 0.921 \text{ ksi (compression)} < 2.7 \text{ ksi} \quad \text{(OK)}$$

  - Stress at bottom of PC girder:

$$f_b = \frac{P_f}{A_{tf}} + \frac{Pe_{tf}}{S_{Btf}} - \frac{(M_{DC1} + M_{DC2})}{S_{Btf}} - \frac{(M_{DC3} + M_{DW})}{S_{BCt}}$$
$$= \frac{614.5}{490.8} + \frac{614.5(15.45)}{4,414} - \frac{(302.4 + 333.7)(12)}{4,414} - \frac{(97.4 + 117.6)(12)}{8,404}$$
$$= 1.367 \text{ ksi (compressive)} < 2.7 \text{ ksi} \quad \text{(OK)}$$

  Note that both the top and bottom fibers are in compression. This satisfies the requirement of no tension for components subjected to permanent loads only per California Amendments Table 5.9.2.3.2b-1 (Caltrans, 2019a).

✓ Load combination PS + Perm + ($LL+IM$)$_{HL\text{-}93}$ (Service I)

  - Compressive stress at top of PC girder:

$$f_{tg} = \frac{P_f}{A_{tf}} - \frac{Pe_{tf}}{S_{Ttf}} + \frac{(M_{DC1} + M_{DC2})}{S_{Ttf}} + \frac{(M_{DC3} + M_{DW})}{S_{TGCt}} + \frac{M_{(LL+IM)HL93}}{S_{TGCt}}$$
$$= \frac{614.5}{490.8} - \frac{614.5(15.45)}{4,414} + \frac{(302.4 + 333.7)(12)}{4,414} + \frac{(97.4 + 117.6)(12)}{28,363} + \frac{967.5(12)}{28,363}$$
$$= 1.331 \text{ ksi (compressive)} < 3.600 \text{ ksi} \quad \text{(OK)}$$

- Compressive stress at top fiber of deck:

  This check is not normally required. The deck resists all loads compositely, so that even with a lower compressive strength, the concrete deck is rarely subjected to significant compressive stress. However, designers may desire to check Service I (PS + Perm + (LL + IM)$_{HL\text{-}93}$) for completeness, applying wearing surface, barrier loads, and HL-93 truck and lane loads to the composite section.

✓ Check tensile stresses at bottom of girder at midspan: (Service III)

This check to prevent cracking at midspan is normally a critical check that can govern the prestressing force and thus area of prestressing strand.

- Load combination PS + Perm + 0.8(LL+IM)$_{HL\text{-}93}$:

$$f_b = \frac{P_f}{A_{tf}} + \frac{Pe_{tf}}{S_{Btf}} - \frac{(M_{DC1} + M_{DC2})}{S_{Btf}} - \frac{(M_{DC3} + M_{DW})}{S_{BCt}} - \frac{0.8\left(M_{(LL+IM)HL93}\right)}{S_{BCt}}$$
$$= \frac{614.5}{490.8} + \frac{614.5(15.45)}{4,414} - \frac{(302.4 + 333.7)(12)}{4,414} - \frac{(97.4 + 117.6)(12)}{8,404} - \frac{0.8(967.5)(12)}{8,404}$$
$$= 0.261 \text{ ksi (compressive)} < 3.6 \text{ ksi} \quad \text{(OK)}$$

In this case, not only is the check satisfied, but the girder remained in compression at the bottom fiber.

- Check concrete stress at harped point (0.4L):

  The concrete stresses at the harp point (0.4L) should also be checked. These can be checked using the same procedures as for midspan. The calculations are not repeated here, but final results are shown.

- Check compressive stresses at 0.4L:

- Load combination PS + Perm
  - f_{tg} = 0.861 ksi (compressive) < 2.7 ksi (OK)
  - f_b = 1.369 ksi (compressive) < 2.7 ksi (OK)

Thus also satisfies the requirement of no tension.

✓ Load combination PS + Perm + *LL*

f_{tg} = 1.309 ksi (compressive) < 3.6 ksi (OK)

f_b = 0.278 ksi (compressive) < 3.6 ksi (OK)

5.3.6.10.4 Fatigue Stress Limit

Although fatigue-related compression in concrete deck slabs with multiple PC girders rarely governs design (because of internal arching action), a check of the compressive stress in the deck for the Fatigue I load combination is required per Article 5.5.3.1. The compression due to the Fatigue I load combination and one half of the sum of effective prestress and permanent loads cannot exceed 0.4 f'_c.

Per C5.5.3.1, the net concrete stress is usually significantly less than the concrete tensile stress limit for cracking, leading to very small steel stress ranges in the prestressing steel less than limiting values.

Fatigue is not likely to control the design and therefore is not checked in this example. Readers are referred to the PCI Bridge Design Manual for more information on how to perform fatigue check.

5.3.6.10.5 Effect of Deck Shrinkage

Article 5.9.3.4.3d shows the procedure to calculate prestress gain due to shrinkage of deck composite section. However, it is Caltrans practice to ignore this prestress gain due to the fact that the deck is designed as reinforced concrete, and cracks are allowed to form in the section. The calculations will not be presented in this example. The readers are referred to the PCI Bridge Design Manual for information and procedure in estimating prestress gain due to deck shrinkage.

5.3.6.11 Design for Strength Limit State

Design of PC girders for the service limit state may produce an adequate design for the strength limit state. However, this must be checked because of the significant additional live load design requirement for the Strength II load combination—the P-15 permit truck.

5.3.6.11.1 Determine Factored Moments

The factored moment at ultimate, M_u, is based on the unfactored moments previously given in Table 5.3.6-9, shown below:

Table 5.3.6-10 Unfactored Bending Moments at 0.4L and Midspan (per Girder)

Location	*M_{DC1} (kip-ft)	*M_{DC2} (kip-ft)	*M_{DC3} (kip-ft)	*M_{DW} (kip-ft)	**$M_{(LL+IM)HL93}$ (kip-ft)	†$M_{(LL+IM)P15}$ (kip-ft)
0.4*L*	290.3	320.3	93.2	112.9	945.8	1,340.5
0.5*L*	302.4	333.7	97.4	117.6	967.5	1,328.9

*Table 5.3.6-3; **Table 5.3.6-4; † Table 5.3.6-5

M_u is taken as the larger of Strength I and II combinations, per Article 3.4.1 Strength I uses the HL-93 design live load, whereas Strength II uses the California P-15 permit truck.

Determine the controlling factored ultimate moment, M_u:

By inspection, moments at midspan govern for Strength I.

- Strength I:

$$M_u = 1.25\left[M_{DC1} + M_{DC2} + M_{DC3}\right] + 1.5M_{DW} + 1.75\left[M_{(LL+IM)HL93}\right]$$

$$M_{u(LL+IM)HL93} = 1.25(302.4 + 333.7 + 97.4) + 1.5(117.6) + 1.75(967.5)$$
$$= 2{,}786.4 \text{ kip-ft/girder}$$

- Strength II:

$$M_u = 1.25\left[M_{DC1} + M_{DC2} + M_{DC3}\right] + 1.5M_{DW} + 1.35\left[M_{(LL+IM)P15}\right]$$

Since live load moment is larger at 0.4L, it is necessary to check both 0.4*L* and 0.5*L* sections to find the critical moments .

- At 0.4*L*:

$$M_{u(LL+IM)P15} = 1.25(290.3 + 320.3 + 93.2) + 1.50(112.9) + 1.35(1{,}340.5)$$
$$= 2{,}858.8 \text{ kip-ft/girder}$$

- At 0.5*L*:

$$M_{u(LL+IM)P15} = 1.25(302.4 + 333.7 + 97.4) + 1.50(117.6) + 1.35(1{,}328.9)$$
$$= 2{,}887.3 \text{ kip-ft /girder} \leftarrow \text{controls}$$

- Strength II governs. M_u = 2,887.3 kip-ft/girder

5.3.6.11.2 Calculate Nominal Flexural Resistance

Based on AASHTO 5.6.3.1 and 5.6.3.2, compute nominal flexural resistance of the section, as shown in Figure 5.3.6-9.

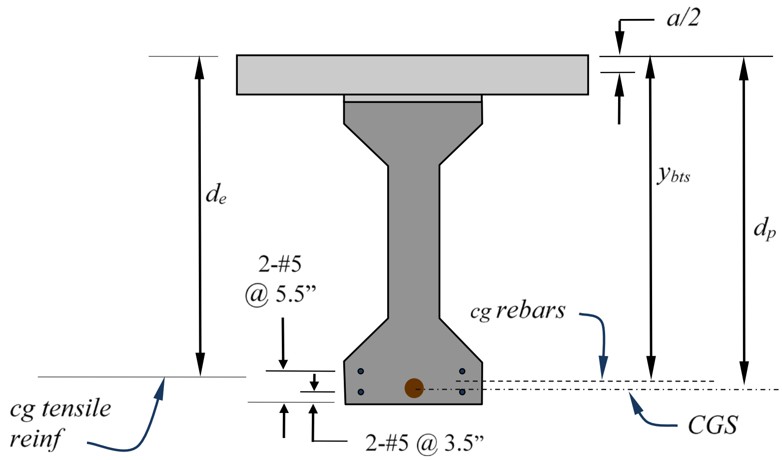


Figure 5.3.6-9 Bridge Section at Midspan

- Determine average prestressing steel stress at ultimate:

  In most applications, the average prestressing steel stress at ultimate can be easily determined from AASHTO Eq. 5.6.3.1.1-1, applicable to typical PC girder sections that use bonded tendons and have an effective stress with $f_{pe} \geq 0.5\ f_{pu}$. If more precise calculations are required, conditions of equilibrium and strain compatibility can be used (AASHTO 5.6.3.2.5).

  For this example, the effective prestress (after all losses),

  f_{pe} = 176.8 ksi > 0.5 f_{pu} = 135 ksi, thus Eq. 5.6.3.1.1-1 is applicable.

  $$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad \text{(AASHTO 5.6.3.1.1-1)}$$

  For low relaxation strand, f_{py}/f_{pu} = 0.9 (Table C5.6.3.1.1-1), thus the value k is determined as follows:

  $$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 2(1.04 - 0.9) = 0.28 \qquad \text{(AASHTO 5.6.3.1.1-2)}$$

  Assuming the compressive stress lies solely within the deck (rectangular stress block develops at ultimate), the neutral axis depth, c, is determined from the following:

$$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s}{0.85 f'_c \beta_1 b + k A_{ps}\left(\frac{f_{pu}}{d_p}\right)} \qquad \text{(AASHTO 5.6.3.1.1-4)}$$

where:

A_{ps} = area of prestressing steel = 3.472 in.^{2}

f_{pu} = specified tensile strength of prestressing steel = 270 ksi

f_{py} = yield strength of prestressing steel = 243 ksi

A_s = area of mild steel tension reinforcement

= four #5 for longitudinal steel requirement = 4 (0.31) = 1.24 in.^{2}

Note that there are four #5 longitudinal bars added to the bottom bulb of the girder as illustrated in Section 5.3.6.16.

A'_s = area of mild steel compression reinforcement = 0

f_s = stress in the mild steel tension reinforcement at nominal flexure resistance

= 60 ksi

f'_s = stress in the mild steel compression reinforcement at nominal flexure resistance

b = effective width of flange in compression (deck) = 72 in.

f'_c = compressive strength in deck concrete at midspan = 4.0 ksi

d_p = distance from extreme compression fiber (deck) to centroid of prestressing tendon = 50 – 4 = 46 in.

From AASHTO 5.6.2.2,

for $f'_c \le 4$ ksi

$\beta_1 = 0.85$

for $f'_c > 4$ ksi

$\beta_1 = 0.85 - 0.05(f'_c - 4) \ge 0.65$

$f'_c = 4.0$ ksi

Therefore, $\beta_1 = 0.85$

c = distance from extreme compression fiber to the neutral axis assuming tendon prestressing steel has yielded (in.)

Assuming that mild reinforcement has also yielded, c is calculated as follows:

$$c = \frac{3.472(270) + 1.24(60)}{0.85(3.6)(0.85)(72) + 0.28(3.472)\left(\frac{270}{46}\right)}$$

$$= 5.24 \text{ in.} < 7 \text{ in. } (\text{deck thickness})$$

Because compressive stresses lie entirely within the slab thickness, the assumption of rectangular section behavior is valid.

$$f_{ps} = 270\left(1 - (0.28)\frac{5.24}{46}\right) = 261.4 \text{ ksi}$$

Determine factored flexural resistance, M_r

M_r = ϕM_n (AASHTO 5.6.3.2.1-1)

where:

M_r = factored flexural resistance

ϕ = resistance factor per Article 5.5.4.2

M_n = nominal flexural resistance

For rectangular sections:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right)$$

where:

d_s = distance from extreme compression fiber (deck) to centroid of mild steel

= 50 – 4.5 = 45.5 in.

a = depth of equivalent rectangular stress block, in.

= $\beta_1 (c)$ = 0.85(5.24) = 4.46 in.

$$M_n = 3.472(261.4)\left(46 - \frac{4.46}{2}\right) + 1.26(60)\left(45.5 - \frac{4.46}{2}\right)$$

$$= 42{,}944 \text{ kip-in.} = 3{,}578.7 \text{ kip-ft}$$

- Determine resistance factor, ϕ:

  According to Figure C5.5.4.2-1 of California Amendments (Caltrans, 2019a) as shown in Figure 5.3.6-10 if the net tensile strain, $\varepsilon_t \geq 0.005$, then the section is defined as tension-controlled and ϕ =1 for flexure.

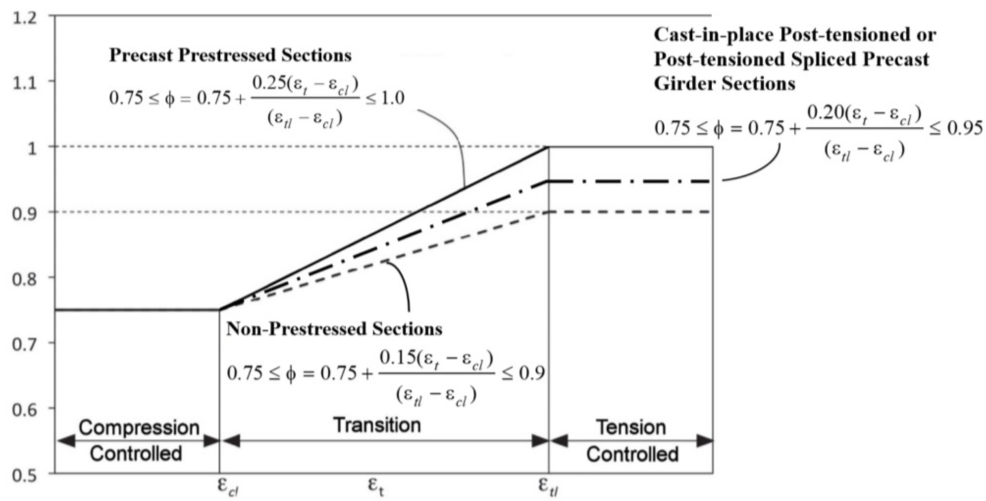


Figure 5.3.6-10 Variation of ϕ with Net Tensile Strain, ε_t, for Grade 60 Reinforcement and Prestressed Members

The net tensile strain is calculated using similar triangles based on the assumed strain distribution through the depth of the section at ultimate, as shown in Figure 5.3.6-11.

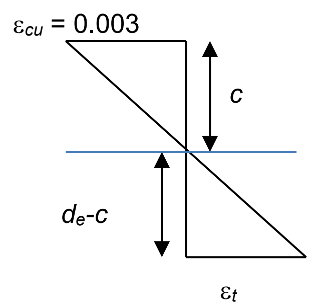


Figure 5.3.6-11 Assumed Strain Distribution through Section Depth at Ultimate

Variables in Figure 5.3.6-11 are defined as follows:

c = distance from extreme compression fiber to the neutral axis (in.)

d_e = effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.) (Aritcle 5.7.2.8-2)

ε_{cu} = failure strain of concrete in compression (in./in.)

ε_t = net tensile strain in extreme tension steel at nominal resistance (in./in.)

$$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y}$$

$$= \frac{3.472(261.4)(46) + 1.24(60)(45.5)}{3.472(261.4) + 1.24(60)} \qquad \text{(AASHTO 5.7.2.8-2)}$$

$$= 45.96 \text{ in.}$$

By similar triangles:

$$\frac{\varepsilon_{cu}}{c} = \frac{\varepsilon_t}{d_e - c}$$

$$\varepsilon_t = \frac{\varepsilon_{cu}}{c}(d_e - c) = \frac{0.003}{5.24}(45.96 - 5.24) = 0.023 \geq 0.005$$

Therefore, the section is tension-controlled, and thus, ϕ = 1.

Check flexural capacity of section:

$$M_r = \phi M_n = 1.0\left(3{,}578.7\right) = 3{,}578.7 \text{ kip-ft } > M_u = 2{,}877.3 \text{ kip-ft} \qquad \left(\text{OK}\right)$$

5.3.6.12 Check Reinforcement Limits

5.3.6.12.1 Maximum Reinforcement

The maximum limit for flexural tension reinforcement to prevent over-reinforced sections, was eliminated in the AASHTO 2006 Interims. The current approach involves reducing the flexural resistance factor when the net tensile strain in the extreme reinforcement is less than 0.005. Although this approach permits sections with less ductility than previous editions if a smaller resistance factor is applied, sections with a net tensile strain less than 0.004 are not recommended because they have reduced ductility and are generally uneconomical. Rather, superstructure members shoud be designed for a net tensile strain of at least 0.004, preferrably 0.005 (for which the resistance factor is 1). The net tensile strain can alternatively be checked, ensuring that the c/d_t (or c/d_e) ratio for the section is not greater than 0.375 (which corresponds to a net tensile strain of 0.005).

5.3.6.12.2 Minimum Reinforcement

To prevent a brittle failure at initial flexural cracking, Article 5.6.3.3 requires that all flexural components have sufficient amount of prestressed and non-prestressed tensile reinforcement to develop a factored flexural resistance, M_r, at least equal to the lesser of: (i) 1.33M_u and (ii) M_{cr} .

where:

M_u = controlling factored moments

M_{cr} = cracking moment as defined in Eqn. 5.6.3.3-1 (kip-in)

For this example,

- The controlling factored moment occurs at midspan due to Strength II combination.

  M_u = 2,887.3 kip-ft

  $1.33M_u$ = 3,840.0 kip-ft

  $$M_{cr} = \gamma_3 \left[\left(\gamma_1 f_r + \gamma_2 f_{cpe} \right) S_c - M_{dnc} \left(\frac{S_c}{S_{nc}} - 1 \right) \right] \qquad \text{(AASHTO 5.6.3.3-1)}$$

  where:

  f_r = modulus of rupture of concrete specified in Article 5.4.2.6

  f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

  M_{dnc} = total unfactored dead load moment acting on the noncomposite section (kip-in.)

  S_c = section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in.^{3})

  S_{nc} = section modulus for the extreme fiber of the noncomposite section where tensile stress is caused by externally applied loads (in.^{3})

  γ_1 = flexural cracking variability factor

  = 1.6 for other than PC segmental structures

  γ_2 = prestress variability factor

  = 1.1 for bonded tendons

  γ_3 = ratio of specified minimum yield strength to ultimate tensile strength of reinforcement

  = 1 for prestressed concrete structures

  $$f_r = 0.24\sqrt{f'_c} = 0.24\sqrt{6} = 0.588 \text{ ksi} \qquad \text{(AASHTO 5.4.2.6)}$$

  $$f_{cpe} = \frac{P_f}{A_g} + \frac{P_f\left(e_c\right)}{S_b} = \frac{556.2}{474} + \frac{556.2(16)}{4{,}770} = 3.039 \text{ ksi}$$

  $$M_{dnc} = M_{DC1} + M_{DC2} = 302.4 + 333.7 = 636.1 \text{ kip-ft} = 7{,}633 \text{ kip-in.}$$

  S_c = S_{BC} = 7,862 in.^{3}

  S_{nc} = S_b = 4,770 in.^{3}

$$M_{cr} = 1\left\{\left[1.6\left(0.558\right)+1.1\left(3.039\right)\right]7{,}862-7{,}633\left(\frac{7{,}862}{4{,}770}-1\right)\right\}$$

$$= 28{,}353 \text{ kip-in.} = 2{,}363 \text{ kip-ft}$$

Since $1.33M_u$ (= 3,849 kip-ft) > M_{cr} (= 2,363 kip-ft) → M_{cr} controls.

ϕM_n = 3,578.7 kip-ft ≥ M_{cr} = 2,363 kip-ft (OK)

5.3.6.13 Design for Shear

Shear design of PC I-girders in this example is performed using the sectional design method of Article 5.7.3 . However, the General Procedure for Shear Design with Tables is used to determine β and θ, per Appendix B5 of AASHTO-CA BDS-8. A design flow chart is provided in Figure CB5.2-5 of Appendix B5 AASHTO-CA BDS-8.

PC girders are designed by comparing the factored shear force (envelope value) and the factored shear resistance at a number of sections along their length, typically at tenth points along the member length and at additional locations near supports. Per Article 5.7.3.3, the shear resistance, V_n, may be taken to consist of the sum of three components:

- Concrete component, V_c, that relies on tensile stresses in the concrete
- Steel component, V_s, that relies on the tensile stresses in the transverse reinforcement
- Prestressing component, V_p, the vertical component of the prestressing force for harped strands

This example illlustrates shear design only at the critical section.

5.3.6.13.1 Determine Critical Section for Shear Design

For the common situation near supports where the reaction force in the direction of the applied shear introduces compression into the end region of a member, Article 5.7.3.2 specifies the location of the critical section for shear to be taken at a distance, d_v, the effective shear depth, from the internal face of the support.

Determine effective shear depth, d_v:

d_v = distance between resultants of tensile and compressive forces due to flexure

= $d_e - a/2$, but not less than the greater of ($0.9d_e$, $0.72h$) (AASHTO 5.7.2.8)

where:

h = h_c, overall depth of composite section

d_e = effective depth from extreme compression fiber to centroid of tensile reinforcement = $h_c - y_{bts}$

where:

y_{bts} = centroid of all tensile reinforcement

a = depth of compression block (taken at midspan for simplicity)

Because harped strands are used, d_e varies near the ends. Thus, either initially assume d_e based on the straight strands or a location equal to 0.05*L* for d_v to calculate d_e.

Using the latter approach:

d_v = 0.05 (70) = 3.5 ft from face of internal support.

The centroid of the tensile reinforcement from the bottom fiber, including both prestressing steel (straight strands only) and mild reinforcement, is calculated based on the following, Figure 5.3.6-12 and Table 5.3.6-11.

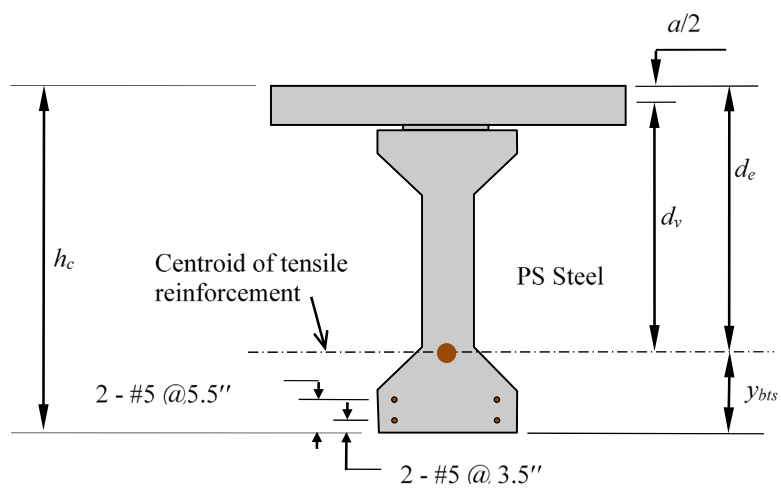


Figure 5.3.6-12 Definitions of y_{bts}, d_e, and d_v at Section Located Near Support

Table 5.3.6-11 Centroid of Tensile Reinforcement

Layer	A_s (in.^{2})	y_i (in.)	$(A_s)(y_i)$ (in.^{3})
1	Two #5 = 0.62	3.5	2.2
2	Two #5 = 0.62	5.5	3.4
3	12(0.217)(270/60) = 11.72	3.5*	41
Total	12.96	–	46.6

*Centroid of 12 straight strands on the tension side.

$$y_{bts} = \frac{46.6}{12.96} = 3.6 \text{ in.}$$

$$d_e = h_c - y_{bts} = 50 - 3.6 = 46.4 \text{ in.}$$

The depth of compression block, a, at d_v can be computed using the procedure presented in the flexure design section. It is found that a = 4.46 in.

$$\frac{a}{2}=\frac{4.46}{2}=2.23\text{ in.}$$

$$y_V=\max\begin{cases}d_e-a/s=46.4-2.2 & =44.2\text{ in.}\\ 0.9d_e\quad=0.9(46.4) & =41.8\text{ in.}\\ 0.7h\quad=0.72(50.0) & =36.0\text{ in.}\end{cases}$$

d_v = 44.2 in.= 3.7 ft

Because 3.7 ft is larger than the initially assumed value of 3.5 ft, it is conservative to simply use the smaller value for d_v (and larger shear) rather than continue iteration. Also, because the bearing pad size has not yet been determined at this stage, it is conservative to assume that the support width equals zero.

Therefore, use d_v = 3.5 ft from centerline of the support.

5.3.6.13.2 Determine Factored Shear Forces

Determine factored shear forces and corresponding factored moment demand at d_v away from the face of the support, which is taken as 3.5 ft from the centerline of the support.

Table 5.3.6-12 Unfactored Shear Forces and Associated Moments at d_v from Face of Support

Shear	V_{DC1}	V_{DC2}	V_{DC3}	V_{DW}	$V_{(LL+IM)HL93}$	$V_{(LL+IM)P15}$
(kip)	15.6	17.2	5	6	65.7	102.2
Associated Moment	M_{DC1}	M_{DC2}	M_{DC3}	M_{DW}	$M_{(LL+IM)HL93}$	$M_{(LL+IM)P15}$
(kip-ft)	57.5	63.4	18.5	22.3	199	304
	Table 5.3.6-3			**Table 5.3.6-4**		**Table 5.3.6-5**

Apply Strength I and Strength II Load Combinations to determine which governs for V_u:

Strength I:

$$\begin{aligned}V_u &= 1.25\left(V_{DC1}+V_{DC2}+V_{DC3}\right)\ +\ 1.5V_{DW}+\ 1.75V_{(LL+IM)HL93}\\ &= 1.25\left(15.6+17.2\ +\ 5\right)\ +\ 1.5\left(6\right)\ +1.75\left(65.7\right)\\ &= 171.2\text{ kip}\end{aligned}$$

Strength II:

$$V_u = 1.25\left(V_{DC1} + V_{DC2} + V_{DC3}\right) + 1.5V_{DW} + 1.35V_{(LL+IM)P15}$$
$$= 1.25(15.6 + 17.2 + 5) + 1.5(6) + 1.35(102.2)$$
$$= 194.2 \text{ kip } \leftarrow \text{controls}$$

Corresponding Strength II factored moment:

$$M_u = 1.25\,[57.5 + 63.4 + 18.5] + 1.5[22.3] + 1.35[304]$$
$$= 618.1 \text{ kip-ft}$$

5.3.6.13.3 Determine Contribution of Concrete

The concrete contribution to shear resistance is determined from the following equation:

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}\,b_v d_v \qquad \text{(AASHTO 5.7.3.3-3)}$$

where:

β = factor indicating the ability of diagonally cracked concrete to transmit tension and shear

b_v = effective web width taken as the minimum web width within the depth, d_v (in.)

d_v = effective shear depth (in.)

For the General Procedure of AASHTO-CA BDS-8 Appendix B5 , the value of β is based on the net longitudinal tensile strain, ε_x, at the middepth of the section for the normal case in which code-minimum transverse reinforcement is provided. This is because such members have the capacity to redistribute shear stresses.

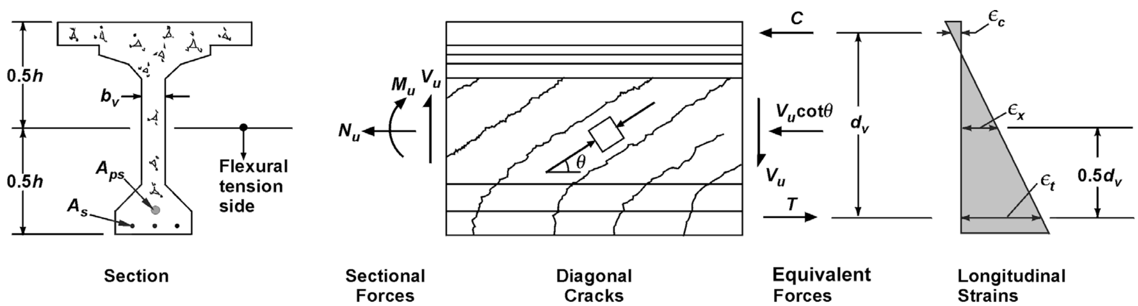


Figure 5.3.6-13 Shear Parameters for Section Containing at Least Minimum Amount of Transverse Reinforcement, V_p = 0

- Determine ε_x:

  For the General Procedure, the longitudinal strain, ε_x, at mid-depth of the section is typically determined using one of the two equations:

  - AASHTO Eq. B5.2-3 when the strain is tensile (positive)
  - AASHTO Eq. B5.2-5 when the strain is compressive (negative)

    The value of 0.5 cotθ may be taken equal to 1 (i.e., θ may be taken as 26.6°) initially during iterations for θ and β, and may also be assumed constant to avoid iterations, without significant loss of accuracy (AASHTO, 2017).

    Note that for some situations (e.g., PC girders made continuous for live load), using Eq. B5.2-3 may be overly conservative when applied near supports because the prestressing strands will be located on the flexural compression side. In such cases, Eq. CB5.2-1 may be used for greater accuracy (AASHTO, 2017). In the case that consideration of torsion is required, V_u shall be replaced by V_{eff} as defined by California amendments Eqs. B5.2-1 and B5.2-2 (Caltrans 2019a).

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5\left|V_u - V_p\right|\cot\theta - A_{ps}f_{po}\right)}{2(E_sA_s + E_pA_{ps})} \qquad \text{(AASHTO, B5.2-3)}$$

where:

$|M_u|$ = absolute value of factored moment corresponding to the factored shear force, not to be taken less than $|V_u - V_p|\, d_v$

= maximum of (618.1 k-ft, $|V_u - V_p|\, d_v$)

V_u = factored shear force = 194.2 kip

V_p = component in the direction of the applied shear of the effective prestressing force; positive if resisting the applied shear (kip)

P_f = total strand force = 560.0 kip (Section 5.3.6.9.2, P_{fg}, non-transformed section)

ω = angle of harped strands, from Figure 5.3.6-14

$$= \tan^{-1}\left(\frac{10-4}{28\times 12}\right) = 1.02^\circ$$

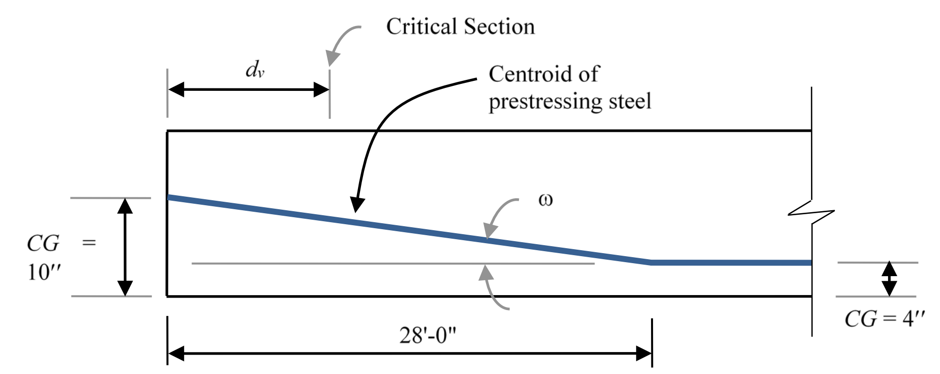


Figure 5.3.6-14 Girder Elevation Near Support and Critical Section for Shear (Ignoring Bearing Pad)

$$V_p = 560.0 \sin\left(1.02^{o}\right) = 10.0 \text{ kip}$$

$$\left|V_u - V_p\right| d_v = \left|194.2 - 10.0\right| 3.5 = 644.7 \text{ kip-ft} > 618.1 \text{ kip-ft}$$

Therefore, | M_u | = 644.7 kip-ft = 7,736 kip-in.

N_u = factored axial force, taken as positive if tensile and negative if compressive = 0 kip

θ = angle of inclination of diagonal compressive stresses

= 26.5° initially assumed, based on taking 0.5 (cot θ) = 1

f_{po} = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete. For the usual levels of prestressing, a value of $0.7f_{pu}$ is appropriate for pretensioned members.

= 0.7 (270) = 189 ksi

A_{ps} = area of prestressing strands on flexural tension side at section

= 3.472 in.^{2}

A_s = 1.24 in.^{2}

$$\varepsilon_x = \frac{\dfrac{\left|7{,}736\right|}{42} + 0.5\left(0\right) + 0.5\left|194.2 - 10.0\right| \cot(26.5) - 3.472\left(189\right)}{2\left[29{,}000\left(1.24\right) + 28{,}500\left(3.472\right)\right]}$$

$$= \frac{\left(-287.3\right)}{269{,}820} = -1.07x10^{-3}$$

Since ε_x is negative at this location at mid-depth, the section is in compression. Therefore, ε_x must be calculated using Eq. B5.2-5, which accounts for the presence of concrete in compression.

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}\right)}{2(E_cA_c + E_sA_s + E_pA_{ps})} \qquad \text{(AASHTO, B5.2-5)}$$

A_c = area of concrete on the flexural tension side

$= 6(19) + 0.5(2)(6)(6) + 7(50/2-6)$

$= 283 \text{ in.}^2$

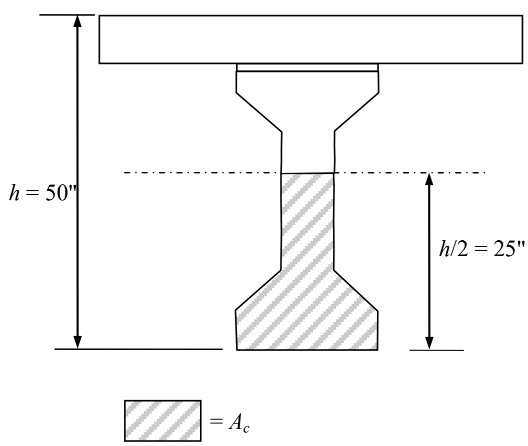


Figure 5.3.6-15 Definition of *A_c*

$$\varepsilon_x = \frac{\left(\frac{|7{,}736|}{42} + 0.5(0) + 0.5|194.2 - 10.0|\cot(26.5) - 3.472(189)\right)}{2\left[4{,}877(283) + 29{,}000(1.24) + 28{,}500(3.472)\right]}$$

$$= \frac{(-287.3)}{3{,}030{,}206} = -0.095x10^{-3}$$

- Determine β and θ:

  For sections with transverse reinforcement equal to or larger than minimum transverse reinforcement, the value of β (factor for concrete shear contribution) and θ (angle of inclination of diagonal compressive stresses) are estimated through iteration from Table B5.2-1 of AASHTO-CA BDS-8 , shown as Table 5.3.6-13. To use this, the ratio (v_u / f'_c) is required in addition to ε_x.

Using ϕ = 0.9 for shear per LRFD Specifications,

$$v_u = \frac{V_u - \phi V_p}{\phi b_v d_v} = \frac{194.2 - 0.9(10.0)}{0.9(7)(42)} = 0.700 \text{ksi}$$

$$\frac{v_u}{f'_c} = \frac{0.700}{6} = 0.117$$

Table 5.3.6-13 Values of θ and β for Sections with Transverse Reinforcement

v_u / f'_c	ε_x × 1,000								
	≤−0.2	≤−0.1	≤−0.05	≤0	≤0.125	≤0.25	≤0.5	≤0.75	≤1
≤0.075	22.3 6.32	20.4 4.75	21.0 4.1	21.8 3.75	24.3 3.24	26.6 2.94	30.5 2.59	33.7 2.38	36.4 2.23
≤0.1	18.1 3.79	20.4 3.38	21.4 3.24	22.5 3.14	24.9 2.91	27.1 2.75	30.8 2.5	34.0 2.32	36.7 2.18
≤0.125	19.9 3.18	21.9 2.99	22.8 2.94	23.7 2.87	25.9 2.74	27.9 2.62	31.4 2.42	34.4 2.26	37.0 2.13
≤0.15	21.6 2.88	23.3 2.79	24.2 2.78	25.0 2.72	26.9 2.60	28.8 2.52	32.1 2.36	34.9 2.21	37.3 2.08
≤0.175	23.2 2.73	24.7 2.66	25.5 2.65	26.2 2.6	28.0 2.52	29.7 2.44	32.7 2.28	35.2 2.14	36.8 1.96
≤0.2	24.7 2.63	26.1 2.59	26.7 2.52	27.4 2.51	29.0 2.43	30.6 2.37	32.8 2.14	34.5 1.94	36.1 1.79
≤0.225	26.1 2.53	27.3 2.45	27.9 2.42	28.5 2.4	30.0 2.34	30.8 2.14	32.3 1.86	34.0 1.73	35.7 1.64
≤0.25	27.5 2.39	28.6 2.39	29.1 2.33	29.7 2.33	30.6 2.12	31.3 1.93	32.8 1.7	34.3 1.58	35.8 1.5

From Table 5.3.6-13 with ε_x = -0.095 $\times 10^{-3}$ and v_u / f'_c = 0.117, the values of θ and β could be determined. Although the values to be selected fall between two choices (boxes) in the table, for hand calculations, it is normally simpler and conservative to use the value of θ in the lower row (larger v_u / f'_c) and value of β in column to the right (larger ε_x) of the computed value in the table.

For this design example, first iteration yields:

θ = 22.8°

β = 2.94

The angle θ was initially assumed to be 26.5°, significantly larger than 22.8°.

Therefore, another iteration is performed using the angle of 22.8°.

$$\varepsilon_x = \frac{\left(\frac{|7{,}736|}{42} + 0.5(0) + 0.5|194.2 - 10.0|\cot(22.8) - 3.472(189)\right)}{2\left[4{,}877(283) + 29{,}000(1.24) + 28{,}500(3.472)\right]}$$

$$= \frac{(-253)}{3{,}030{,}206} = -0.083 \times 10^{-3}$$

From Table 5.3.6-13, Iteration 2 yields the same values for θ and β. Therefore, no further iteration is required and the following values are used in design at this section:

θ = 22.8°

β = 2.94

- Compute concrete contribution to shear resistance, V_c :

$$V_c = 0.0316\beta\sqrt{f_c'}b_v d_v$$
$$= 0.0316(2.94)\sqrt{6}(7)(42) = 66.9 \text{ kip} \qquad \text{(AASHTO 5.7.3.3-3)}$$

5.3.6.13.4 Requirement for Transverse Reinforcement

Check if shear reinforcement is required, i.e., when $V_c \geq 0.5\phi\ (V_c - V_p)$

V_u = 194.2 kip > 0.5 (0.9) (66.9 + 10.0) = 34.6 kip

Therefore, transverse shear reinforcement is required at the critical section.

The required area of transverse reinforcement is based on satisfying the following design relationship:

$$\frac{V_u}{\phi} \leq V_n = V_c + V_p + V_s \qquad \text{(AASHTO 5.7.3.3-1)}$$

Solving this equation for V_s leads to:

$$V_s = \frac{V_u}{\phi} - V_c - V_p$$

Therefore, the required contribution of the transverse reinforcement, V_s, is:

$$V_s = \frac{194.2}{0.9} - 66.9 - 10.0 = 139 \text{ kip}$$

The required area of transverse reinforcement can conveniently be expressed in design as an area per length, i.e., (A_v/s) based on rearrangement of AASHTO Eq.

5.7.3.3-4:

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s}$$

$$\frac{A_v}{s} = \frac{V_s}{f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}$$

where:

s	=	spacing of transverse reinforcement measured in a direction parallel to the longitudinal reinforcement
A_v	=	area of shear reinforcement within a distance s
θ	=	angle of inclination of diagonal compressive stresses
α	=	angle of inclination of transverse reinforcement to longitudinal axis
	=	90° for vertical stirrups
f_y	=	yield strength of transverse reinforcement

$$\frac{A_v}{s} = \frac{139}{60(42)(\cot 22.8^\circ + \cot 90^\circ)\sin 90^\circ}$$

$$= 0.023 \frac{\text{in.}^2}{\text{in.}}$$

Using #5 double-leg stirrups for transverse reinforcement,

A_v = 0.31 (2) = 0.62 in.^{2}

$$\text{Spacing, } s = \frac{0.62}{0.023} = 27 \text{ in.}$$

Using # 5 double-leg stirrups at 12 in. on center (s = 12 in.) near supports.

Note: Larger spacing, up to the maximum permitted by AASHTO-CA BDS-8, may be selected for section beyond 4 ft at the discretion of the designer.

This corresponds to a contribution of transverse reinforcement, V_s, to nominal shear resistance:

$$V_{s,provided} = \frac{0.62(60)(42)(\cot 22.8^\circ)}{12} = 309.7 \text{ kip}$$

$V_n = V_c + V_p + V_s$

$$V_n = 66.9 + 10.0 + 309.7 = 386.6 \text{kip} > \frac{V_u}{\phi} = 215.8 \text{ kip} \qquad \text{(OK)}$$

5.3.6.13.5 Determine Maximum Spacing for Transverse Reinforcement

Per Article 5.7.2.6 , the spacing of transverse reinforcement, s, cannot exceed the maximum permissible spacing, s_{max} (i.e., $s \le s_{max}$). The maximum spacing, s_{max}, depends on the level of shear stress, v_u. From previous calculation,

$$v_u = \frac{\left|V_u - \phi V_p\right|}{\phi b_v d_v} = 0.700 \text{ ksi}$$

If $v_u < 0.125 f'_c$, $s_{max} = 0.8\ d_v \le 18$ in. (CA 5.7.2.6-1)

$0.125 f'_c = 0.125\ (6) = 0.75$ ksi > $v_u = 0.7$ ksi

$s_{max} = 0.8\ d_v = 33.6$ in.

$s_{max} = 18$ in. ← controls

Therefore, $s_{max} = 18$ in.

Spacing provided $s = 6$ in. < 18 in. (OK)

Note that tighter spacing per Eq. 5.7.2.6-2 applies for cases in which $v_u \ge 0.125 f'_c$.

5.3.6.13.6 Check Minimum Transverse Reinforcement

The area of transverse reinforcement, A_v, provided cannot be less than that required by Eq. 5.7.2.5-1:

$$A_v \ge 0.0316 \frac{\sqrt{f'_c}\, b_v s}{f_y}$$ (AASHTO 5.7.2.5-1)

For $s = 12$ in. as provided:

$$A_v = 0.62 \text{ in.}^2 > 0.0316\sqrt{6}\,\frac{7(12)}{60} = 0.108 \text{ in.}^2$$

Therefore, #5 double-leg stirrups at 12 in. on center satisfy the minimum transverse reinforcement requirement.

5.3.6.13.7 Check Maximum Nominal Shear Resistance

To ensure that the web concrete will not crush prior to yielding of transverse reinforcement, Article 5.7.3.3 requires that the nominal shear resistance, V_n, be limited to the smaller of Eq. 5.7.3.3-1 and Eq. 5.7.3.3-2:

$V_n = V_c + V_s + V_p = 386.6$ kip (AASHTO 5.7.3.3-1)

$V_n = 0.25\ f'_c\, b_v\, d_v + V_p = 0.25\ (6)(7)(42) + 10.0 = 451$ kip (AASHTO 5.7.3.3-2)

Therefore, the nominal shear resistance is 386.6 kip.

Using the above procedure, the transverse reinforcement along the entire girder can be similarly determined.

5.3.6.14 Design for Interface Shear Transfer between Girder and Deck

The interface shear transfer between precast concrete girder and cast-in-place deck shall be designed according to Article 5.7.4. The interface resistance shall satisfy:

$$V_{ri} = \phi V_{ni} \geq V_{ui} \qquad \text{(AASHTO 5.7.4.3-1 \& 5.7.4.3-2)}$$

where:

V_{ri} = factored interface shear resistance (kip)

V_{ni} = nominal interface shear resistance (kip)

V_{ui} = factored interface shear (kip)

ϕ = resistance factor = 0.9

Using AASHTO Eq. (C5.7.4.5-7), the factored interface shear can be taken as:

$$V_{hi} = V_{ui} = \frac{V_u}{d_v}$$

where:

V_u = factored vertical shear under Strength Limit State

d_v = effective depth for shear

All sections along the entire length of the girder are required to satisfy the Article 5.7.4 requirement. For this example, the interface shear design is only demonstrated at the d_v (= 42 in.) location from face of support.

From Section 5.3.6.13.2, the factored shear V_u = 194.2 kip at d_v. The factored interface shear,

$$V_{ui} = \frac{194.2}{42} = 4.62 \text{ kip/in.}$$

$$V_{ni} = cA_{cv} + \mu\left(A_{vf}f_y + P_c\right) \qquad \text{(AASHTO 5.7.4.3-3)}$$

$$A_{cv} = b_{vi}\,L_{vi} \qquad \text{(AASHTO 5.7.4.3-6)}$$

where:

c = cohesion factor from Article 5.7.4.4 (ksi)

μ = friction factor from Article 5.7.4.4 (ksi)

A_{cv} = area of concrete engaged in interface shear transfer (in.^{2})

A_{vf} = area of interface shear reinforcing crossing the shear plane within A_{cv} (in.^{2})

f_y = yield stress of interface shear reinforcement (ksi)

P_c = permanent compressive force (kip)

b_{vi} = interface width (in.)

L_{vi} = interface length (in.)

For CIP concrete slab on clean concrete girder surfaces, free of laitance and with surface roughened to an amplitude of 0.25 in.:

c = 0.28 ksi (AASHTO 5.7.4.4)

μ = 1 (AASHTO 5.7.4.4)

A_{cv} = 19 (1) = 19 in.^{2}

The amount of vertical shear reinforcement provided at d_v is #5 bar, double legs, at 6 in. spacing (Sect. 5.3.6.13.4), acting as interface shear reinforcement extended into the deck.

$$A_{vf} = \frac{0.31}{6} = 0.052\,\frac{\text{in.}^2}{\text{in.}}$$

V_{ni} = 0.28(19) + 1[(0.052)(60) + 0] = 8.44 kip/in.

ϕV_{ni} = 0.9 (8.44) = 7.6 kip/in. > V_{ui} = 4.62 kip/in. (OK)

5.3.6.14.1 Check Minimum Interface Shear Reinforcement

The minimum interface shear reinforcement required is

$$A_{vf,min} = \frac{0.05A_{cv}}{f_y} = \frac{0.05(19)}{60} = 0.016\,\frac{\text{in.}^2}{\text{in.}}$$

Provide $A_{vf} = 0.052\,\frac{\text{in.}^2}{\text{in.}} > 0.016\,\frac{\text{in.}^2}{\text{in.}}$

Therefore, minimum interface shear reinforcement requirement is met.

5.3.6.14.2 Check Maximum Nominal Shear Resistance

The maximum nominal shear resistance used in design shall be the lesser of the two:

- $V_{ni} \le K_1 f'_c A_{cv}$ (AASHTO 5.7.4.3-4)
- $V_{ni} \le K_2 A_{cv}$ (AASHTO 5.7.4.3-5)

where:

f'_c = concrete compressive strength of deck slab (ksi)

K_1 = fraction of concrete strength available to resist interface shear

K_2 = limiting interface shear resistance

For CIP concrete slab on clean concrete girder surfaces, free of laitance with surface roughened to an amplitude of 0.25 in.:

K_1 = 0.3 (AASHTO 5.7.4.4)

K_2 = 1.8 ksi for normal weight concrete (AASHTO 5.7.4.4)

- $V_{ni} \le 0.3\ (4.0)\ (19) = 22.8$ kip/in. ← controls
- $V_{ni} \le 1.8\ (19) = 34.2$ kip/in.

The V_{ni} provided = 8.44 kip/in. < 22.8 kip/in. (OK).

5.3.6.15 Check Minimum Longitudinal Reinforcement

The minimum longitudinal reinforcement (including both prestressed and non-prestressed reinforcement on the flexural tension side) at all locations along the girder shall be proportioned to satisfy:

$$A_{ps}f_{ps} + A_s f_y \ge \frac{|M_u|}{d_v \phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_c} - V_p\right| - 0.5V_s\right)\cot\theta \qquad \text{(AASHTO 5.7.3.5-1)}$$

At d_v from the face of the support:

M_u = 618.1 kip-ft

V_u = 194.2 kip

V_s = 309.7 kip, but not to exceed $\frac{V_u}{\phi_v}$

where:

$$\frac{V_u}{\phi_v} = \frac{194.2}{0.9} = 215.8 \quad \text{kip}$$

Therefore, V_s = 218.5 kip

V_p = 10.0 kip

N_u = 0 kip

d_v = 42 in.

θ = 22.8°

f_{ps} = f_{pe} = 161.3 ksi (from Section 5.3.6.9.2)

The determination of minimum longitudinal reinforcement at d_v from face of support is illustrated below.

$$\frac{|M_u|}{d_v \phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta$$

$$= \frac{\left|618.2(12)\right|}{(42)(1.0)} + 0.5\left(\frac{0.0}{1.0}\right) + \left(\left|\frac{194.2}{0.9} - 10.0\right| - 0.5(218.5)\right)\cot 22.8^{o} = 406.4\,\text{kip}$$

Transfer length, L_t, from girder end is taken as 30(strand diameters) = 30(0.6) = 18 in. This is less than the distance d_v from the end. Therefore, the strands have developed the full prestressing force and the effective prestress f_{ps} = 161.3 ksi is used.

A_{ps} = area of 12 straight strands = 12(0.217) = 2.604 in.^{2}

A_s = area of four #5 rebar = 4(0.31) = 1.24 in.^{2}

$A_{ps} f_{ps} + A_s f_y$ = 2.604(161.3) + 1.24(60) = 494.4 kip > 409.7 kip

Therefore, the minimum reinforcement requirement is satisfied.

Note that the minimum reinforcement requirement needs to be satisfied in all locations along the girder. AASHTO Eq. 5.7.3.5-2 must also be satisfied at the inside edge of the bearing area of simple end supports to the section of critical shear.

5.3.6.16 Determine Pretensioned Anchorage Zone Reinforcement

- Splitting resistance:

  Article 5.9.4.4.1 requires the following vertical reinforcement be provided within the distance $h/4$ from the end of the girder to provide splitting resistance to bursting stresses.

  $P_r = f_s A_s$ (AASHTO 5.9.4.4.1-1)

  where:

  f_s = stress in steel not to exceed 20 ksi

  A_s = total area of vertical reinforcement located within the distance $h/4$ from end of beam (in.^{2})

  h = overall dimension (height) of the precast I girder in the direction in which splitting resistance is being evaluated (in.)

  P_r = factored bursting resistance of pretensioned anchorage zone provided by transverse reinforcement (kip), not less than 4% of prestressing force at transfer, P_i

  P_r = 0.04 (P_i) = 0.04 (0.75) (270) (3.472) = 28.1 kip

  $$A_s = \frac{28.1}{20} = 1.41\,\text{in.}^2$$

  Using #5 bars with 2 vertical legs,

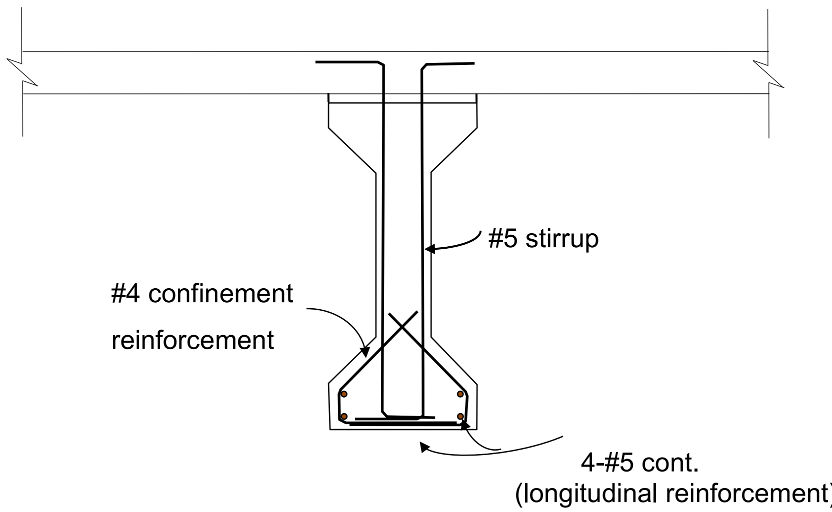
  $$\text{Number of bars required} = \frac{1.41}{0.31(2)} = 2.3$$

  Therefore, use three #5 double legs within $h/4$ (42/4 = 10.5 in.) from end of girder.

- Confinement reinforcement:

  Article 5.9.4.4.2 requires reinforcement be placed to confine the prestressing steel in the bottom flange, over the distance 1.5d from the end of the girder, using #4 rebar with spacing not to exceed 6 in. and shaped to enclose the strands.

  Place #4 rebar at no more than 6 in. on center over the distance 1.5d = 1.5(42) = 63 in. (conservative) from the end of the girder to confine the prestressing strands in the bottom flange.



Note: See XS Sheets for standard reinforcement (not shown).

Figure 5.3.6-16 Transverse, Longitudinal, and Confinement Reinforcement Details

5.3.6.17 Calculate Deflection and Camber

The following three aspects of deflection and camber are addressed in this design example:

- Determine and specify unfactored instantaneous girder deflections due to deck and rail for plan sheets
- Check live load deflection against AASHTO-CA BDS-8 deflection criteria
- Determine and specify minimum haunch thickness at supports for plan sheets

Total deflection of the girder is estimated as the sum of the short-term and long-term deflections. Short-term deflections are immediate and are based on an estimate of the modulus of elasticity and the effective moment of inertia. Girder and deck slab self-weight are carried non-compositely by the girder alone, while dead loads such as barriers and overlays as well as live loads are carried by the composite girder-deck system. Long-term deflections consist of long-term deflections at erection and long-term deflection at final stage (may be assumed to be approximately 20 years).

5.3.6.17.1 Calculate Girder Deflections due to Deck and Rail

In this section, the instantaneous, unfactored girder camber and deflections due to prestressing force and self-weight of the deck, haunch, barrier, and future wearing surface are calculated for the contract plans.

- Initial camber due to prestressing force at midspan can be estimated using case (4) of Table 5.3.4-1. After simplifying, the deflection, Δ_p, is expressed as,

$$\Delta_p = \frac{P_i}{E_{ci} I}\left(\frac{e_c L^2}{8} - \frac{e'(bL)^2}{6}\right)$$

where:

P_i = total prestressing force immediately in prestress strands after transfer (kip)

= $(P_j - ES)A_{ps}$ = (202.5 – 15.72) (3.472) = 648.5 kip

E_{ci} = modulus of elasticity of concrete at initial (ksi) = 4,531 ksi

I = initial gross (non-transformed) moment of inertia of the girder (in.^{4})

= 95,400 in.^{4}

e_c = eccentricity of prestressing strands at midspan (in.) = 20 – 4 = 16 in.

e' = difference between eccentricity of prestressing strands at midspan and at end of girder (in.) = 10 – 4 = 6 in.

L = girder length = 71(12) = 852 in.

bL = distance from end of girder to harped point (in.) = 28(12) = 336 in.

$$\Delta_p = \frac{648.5}{4{,}531(95{,}400)}\left(\frac{16.0(852)^2}{8} - \frac{6.0(336)^2}{6}\right) = 2.00 \text{ in.} \quad \text{(upward)}$$

- Immediate deflection due to girder self-weight at midspan:

The equation for deflection of a simply supported girder with a distributed load:

$$\Delta_g = \frac{5}{384}\left(\frac{w_g L^4}{E_{ci} I}\right)$$

where:

w_g = distributed weight of the girder = 0.494 kip/ft = 0.041 kip/in.

Deflection due to beam self weight to be used in computing deflection at erection (with span = 70 ft = 840 in.),

$$\Delta_g = \frac{5}{384}\left(\frac{0.041(840)^4}{4{,}531(95{,}400)}\right) = 0.62 \text{ in.} \qquad \text{(downward)}$$

- Immediate deflection due to weight of deck and haunch at midspan (non-composite section):

  w_s = 0.525 + 0.02 = 0.545 kip-ft = 0.045 kip-in.

  E_c = 4,877 ksi

  $$\Delta_s = \frac{5}{384}\left(\frac{0.045(840)^4}{4{,}877(95{,}400)}\right) = 0.63 \text{ in.} \qquad \text{(downward)}$$

- Immediate deflection due to barrier weight at midspan (composite section):

  w_{br} = 0.159 kip-ft = 0.013 kip-in.

  E_c = 4,877 ksi, and I = 259,449 in.^{4} (gross composite section)

  $$\Delta_{br} = \frac{5}{384}\left(\frac{0.013(840)^4}{4{,}877(259{,}449)}\right) = 0.07 \text{ in.} \qquad \text{(downward)}$$

- Immediate deflection due to future wearing surface at midspan (composite section):

  w_{fw} = 0.192 kip-ft = 0.016 kip-in.

  E_c = ksi, and I = 259,449 in.^{4} (gross composite section)

  $$\Delta_{fw} = \frac{5}{384}\left(\frac{0.016(840)^4}{4{,}877(259{,}449)}\right) = 0.08 \text{ in.} \qquad \text{(downward)}$$

- Immediate deflections for plans (deck and haunch; barrier rails and future wearing surface):

  The following unfactored instantaneous girder deflection values at midspan should be shown on the contract plans.

  Deck: Unfactored instantaneous girder deflection due to deck and haunch = Δ_s = 0.63 in.

  Rail: Unfactored instantaneous girder deflection due to barrier rail and future wearing surface = $\Delta_{br} + \Delta_{fw}$ = 0.07 + 0.08 = 0.15 in.

5.3.6.17.2 Compare Live Load Deflection to Article 2.5.2.6.2 Limit

Girder live load deflection check is estimated using composite section properties and concrete strength at service, and compared to the limit as specified in Article 2.5.2.6.2. It should be noted that the deflection criteria in Article 2.5.2.6.2 is optional for California bridges. However, for specific situations, such as bridge widening where the deflection may impair the minimum vertical clearance, the deflection must be accounted for in the design.

The instantaneous live load deflection for a simple span bridge occurs at midspan due to the HL-93 truck axles placed in the location shown below in Figure 5.3.6-17, together with the HL-93 lane load (not shown).

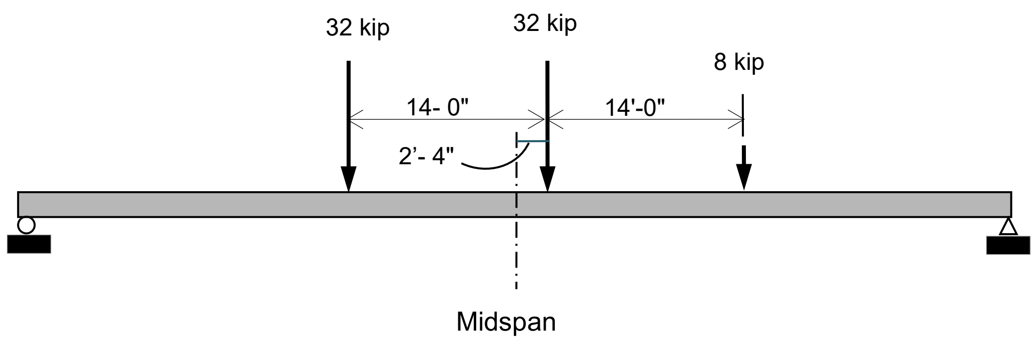


Figure 5.3.6-17 Position of Truck to Produce Maximum Moment

The live load for each lane can be obtained from any structural analysis software program, such as CT Bridge. The deflection for each girder is calculated by multiplying the deflection per lane by the ratio of (number of lanes/number of girders). This ratio can also be estimated using the moment distribution factor (DFM) shown in Sec. 5.3.6.7.3, which is the simpler way.

Since the deck concrete and girder concrete differ in strength, transformed properties are used to calculate the live load deflection. As shown in Sec. 5.3.6.7.3, the DFM for this bridge is 0.463.

$$\Delta_{LL} = DFM\,(\Delta_{LL\ per\ lane}) = 0.463\,(\Delta_{LL\ per\ lane})$$

From structural analysis software, $\Delta_{LL\ per\ lane}$ = 1.01 in. (downward)

$$\Delta_{LL} = 0.463\,(1.01) = 0.47 \text{ in. (downward)}$$

This instantaneous live load deflection is compared to the AASHTO LRFD recommended limitation of L/800 for general vehicular loading.

$$\frac{L}{800} = \frac{70(12)}{800} = 1.05 \text{ in.}$$

The live load deflection is less than the AASHTO limit and therefore acceptable.

5.3.6.17.3 Determine Minimum Haunch Thickness

The minimum haunch thickness at supports is intended to help ensure that the specified haunch at midspan is achieved in the field. (See Section 5.3.4.5.1 for discussion on importance of the haunch.)

The midspan haunch thickness is first specified by the designer (note that this is a recommended minimum value). Based on this, the designer would calculate the minimum haunch thickness at the supports. Both the minimum haunch thickness at the supports and the designer-specified midspan haunch thickness should be shown on the plans.

At erection, the net upward camber of the girder at midspan due to prestressing force plus self-weight, $\Delta_{g,erect}$, is calculated based on the instantaneous deflection components multiplied by the PCI factors. These factors account for the time-dependent effects between girder casting and erection.

$$\Delta_{g,erect} = \left(MPCI_{erect}\Delta_p - MPCI_{erect}\Delta_g\right)$$

where:

$\Delta_{g,erect}$ = camber at midspan at erection, due to long-term effects of prestressing force and girder self-weight (in.)

$MPCI_{erect}$ = PCI multiplier for long-term effects at erection (Table 5.3.4-2)

Δ_p = immediate camber of girder at transfer due to prestressing force (in.)

Δ_g = immediate deflection of girder at midspan at transfer due to self -weight (in.)

As shown in Figure 5.3.6-18, the minimum haunch thickness at the supports, TH_{sup}, is then calculated as the difference (at the centerline of the girder) between the long-term camber at midspan at erection, $\Delta_{g,erect}$ and the instantaneous deflection of the girder at midspan due to the weight of the deck and haunch, plus the designer-specified haunch at midspan, TH_{mid}.

$$TH_{sup} = \Delta_{g,erect} - \Delta_s + TH_{mid}$$

where:

TH_{sup} = haunch thickness at supports

$\Delta_{g,erect}$ = camber at midspan at erection, due to long-term effects of prestressing force and girder self-weight (in.)

Δ_s = immediate deflection at midspan due to deck weight (in.)

TH_{mid} = designer-specified haunch thickness at midspan (in.)

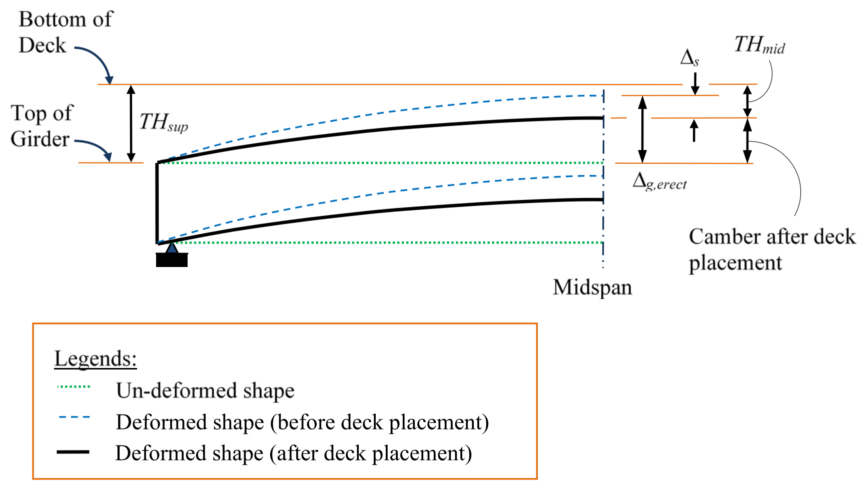


Figure 5.3.6-18 Relationship between Specified Haunch Thickness at Midspan and Minimum Haunch Thickness at Supports

For this example:

$$\Delta_{g,erect} = 1.8\Delta_p - 1.85\Delta_g = 1.8\,(2.00) - 1.85\,(0.62) = 2.45 \text{ in. (upward)}$$

Specifying a haunch thickness of 1 in. at midspan, the minimum haunch thickness required at the supports is determined as follows:

$$TH_{sup} = \Delta_{g,erect} + \Delta_s + TH_{mid} = 2.45 + (-0.63) + 1 = 2.82 \text{ in.}$$

Therefore, the following should be specified on contract plans:

Haunch thickness at midspan: 1 in.

Minimum haunch thickness at supports: 3 in.

Note that the deck cross slope and girder top flange width do not affect the specified midspan haunch thickness. For situations where the cross slope is relatively large and/or the top flange is very wide (such as for wide-flange girders), thickness of the haunch at midspan on both sides of the girder flange must be carefully considered.

NOTATION

a = depth of equivalent rectangular compression stress block (in.)

A_c = concrete area of composite section

A_{cv} = area of concrete engaged in interface shear transfer (in.^{2})

A_g = gross area of girder section (in.^{2})

A_i = area of individual component (in2)

A_{ps} = area of prestressing steel (in^2)

A_s = area of non-prestressed tension reinforcement (in.^{2})

A'_s = area of compression reinforcement (in.^{2})

A_{tf} = area of transformed section, at final (in.^{2})

A_{ti} = gross area of girder concrete at time of force transfer (in.^{2})

A_v = area of transverse reinforcement within distance s (in.^{2})

A_{vf} = area of interface shear reinforcing crossing the shear plane within A_{cv} (in.^{2})

ADL = added dead load (kip)

b = width of the compression face of a member (in.)

b_{eff} = effective flange width (in.)

bL = distance from end of girder to harped point (in.)

b_v = effective web width taken as the minimum web width, measured parallel to the neutral axis, between resultants of the tensile and compressive forces due to flexure; this value lies within the depth, d_v (in.)

b_{vi} = interface width (in.)

c = distance from extreme compression fiber to the neutral axis (in.); also cohesion factor from AASHTO 5.7.4.4 (ksi)

cg = center of gravity

CGC = center of gravity of the concrete section

CGS = center of gravity of the strands

D = structure depth (ft) or height of standard shape of PC girder (in.)

DC = weight of supported structures (kip)

d_b = nominal strand diameter (in.)

d_e = effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.)

DF_{DL} = dead load distribution factor

DFM = live load moment distribution factor

DFV = live load shear distribution factor

d_p = distance from extreme compression fiber to the centroid of the prestressing tendons (in.)

d_s = distance from extreme compression fiber to the centriod of the nonprestressed tensile reinforcement (in.)

D_s = superstructure depth (ft)

d_t = distance from extreme compression fiber to the centroid of tensile reinforcement

d_v = the effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure (in.)

DW = superimposed dead load (kip)

e = eccentricity of the anchorage device or group of devices with respect to the centroid of the cross section; always taken as positive (in); also the base of Napierian logarithms

e' = difference between eccentricity of prestressing steel at midspan and at end (in.)

E_B, E_c = modulus of elasticity of girder material (ksi)

e_c = eccentricity of strands measured from center of gravity of girder at midspan (in.)

E_{ci} = modulus of elasticity of concrete at initial time (ksi)

E_{ct} = modulus of elasticity of concrete at transfer or time of load application (ksi)

E_D = modulus of elasticity of deck material (ksi)

e_g = distance between centers of gravity of girder and deck (in.)

e_m = eccentricity at midspan (in.)

E_p, E_{ps} = modulus of elasticity of prestressing tendons (ksi)

E_s = modulus of elasticity of mild reinforcing steel (ksi)

e_{tf} = distance between centers of gravity of strands and concrete section at time of service (in.)

e_{ti} = distance between centers of gravity of strands and concrete section at time of transfer (in.)

f_b = concrete flexural stress at extreme bottom fiber (ksi)

f'_c = specified compressive strength of concrete used in design (ksi)

f'_{ci} = specified compressive strength of concrete at time of initial loading or prestressing (ksi); nominal concrete strength at time of application of tendon force (ksi)

f_{cgp} = concrete stress at the center of gravity of prestressing tensonds that results from the prestressing force at either transfer or jacking and the self-weight of the member at sections maximum moment (ksi)

f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

f_g = stress in the member from dead load (ksi)

f_{pbt} = stress in prestressing steel immediately prior to transfer (ksi)

f_{pe} = effective stress in the prestressing steel after losses (ksi)

f_{pi} = prestressing steel stress immediately prior to transfer (ksi)

f_{pj} = stress in prestressing steel at initial jacking (ksi)

f_{po} = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi)

f_{ps} = average stress in prestressing steel at the time for which the nominal resistance is required (ksi)

f_{pu} = specified tensile strength of prestressing steel (ksi)

f_{py} = yield strength of prestressing steel (ksi)

f_r = modulus of rupture of concrete (ksi)

f_s = stress in mild tension reinforcement at nominal flexural resistance (ksi) (5.3.4.6.1)

f'_s = stress in the mild steel compression reinforcement at nominal flexure resistance (ksi)

F_T = required tension capacity provided by reinforcement (kip)

f_{tg} = concrete stress at top of precast girder (ksi)

f_y = yield strength of mild steel (ksi)

f'_y = specified minimum yield strength of compression reinforcment (ksi)

H = average annual ambient mean relative humidity (percent)

h = web dimension of PC girder (in.)

h_c = overall depth of composite section (in.)

I = initial gross (non-transformed) moment of inertia of the girder (in.^{4})

I_c = moment of inertia of composite section about centroidal axis, neglecting reinforcement (in.^{4})

I_{cg}, I_g = moment of inertia of the girder concrete section about the centroidal axis, neglecting reinforcement (in.^{4})

I_{Ct} = moment of inertia of composite transformed section (in.^{4})

I_e = effective moment of inertia (in.^{4})

I_g = gross moment of inertia of girder (in.^{4})

I_{ti} = moment of inertia of concrete section at initial stage, transformed (in.^{4})

K_1 = fraction of concrete strength available to resist shear

K_2 = limiting interface shear resistance (ksi)

K_g = longitudinal stiffness parameter (in.^{4})

L = span length or girder length (ft)

LL = live load (kip)

M_{cr} = cracking moment (kip-in.)

M_{DC1} = moment due to self-weight of girder (kip-ft)

M_{DC2} = moment due to self-weight of deck and haunch (kip-ft)

M_{DC3} = moment due to self-weight of barrier (kip-ft)

M_{dnc} = total unfactored dead load moment action on the monolithic or noncomposite section (kip-ft)

M_{DW} = moment due to future wearing surface (kip-ft)

M_g = midspan moment due to self-weight of girder (kip-ft)

M_{HL93} = moment due to enveloped HL-93 trucks (kip-ft)

M_{LL} = moment due to live loads (kip-ft)

M_n = nominal flexure resistance (kip-in.)

M_{P15} = moment due to enveloped P15 truck (kip-ft)

$MPCI_{erect}$= PCI Multipliers for camber/deflection at time of erection

M_r = factored flexural resistance of a section in bending (kip-in.)

M_{slab} = moment due to weight of deck slab (kip-ft)

M_u = controlling factored moments (kip-ft)

M_x = moment at location x (kip-ft)

n = modular ratio between beam and deck

N_b = number of beams, stringers, or girders

N_u = factored axial force, taken as positive if tensile and negative is compressive (kip)

P, P_e = Effective force in prestress strands after all losses (kip)

P_c = permanent compressive force (kip)

P_f = force in prestress strands after losses (kip)

P_{fg} = effective force in prestress strands after all losses for gross section design (kip)

P_i = force in prestress strands after elastic shortening loss (kip)

P_j = force in prestress strands before losses (kip)

P_r = factored bearing resistance of anchorages (kip)

r = radius of gyration(in.)

S = spacing of girders or webs (ft)

s = spacing of reinforcing bars (in.)

S_b = section modulus for the bottom extreme fiber of the girder where tensile stress is caused by externally applied loads (in.^{3})

S_{BC} = section modulus for the bottom extreme fiber of the composite section where tensile stress is caused by externally applied loads (in.^{3})

S_{BCt} = section modulus for the bottom extreme fiber of the composite section - transformed (in. ^{3})

S_{Btf} = section modulus for the bottom extreme fiber of the composite section - transformed, at service stage (in. ^{3})

S_c = section modulus for the extreme fiber of the composite sections where tensile stress is caused by externally applied loads (in.^{3})

S_{nc} = section modulus for the extreme fiber of the monolithic or noncomposite sections where tensile stress is caused by externally applied loads (in.^{3})

S_t = section modulus for the top extreme fiber of the sections where tensile stress is caused by externally applied loads (in.^{3})

S_{tc} = section modulus for the top extreme fiber of the composite sections where tensile stress is caused by externally applied loads (in.^{3})

S_{TDCt} = section modulus for the top extreme fiber of the composite sections at top of deck level, at service, transformed (in.^{3})

S_{TGCt} = section modulus for the top extreme fiber of the composite sections at top of girder level, at service, transformed (in.^{3})

S_{Ttf} = section modulus for the top extreme fiber of the composite sections at top of girder level, at service, transformed (in.^{3})

S_{Tti} = section modulus for the top extreme fiber precast girder, at initial, transformed (in.^{3})

T = tensile stress in concrete (ksi)

t_s = thickness of concrete deck slab (in.)

t_h = haunch thickness at midspan (in.)

TH_{mid} = haunch thickness at midspan (in.)

TH_{sup} = haunch thickness at support (in.)

w = uniform dead load (klf)

w_{br} = uniform dead load−weight of barrier (klf)

w_{fw} = uniform dead load−weight of future wearing surface (klf)

w_g = uniform dead load−weight of girder (klf)

w_h = uniform dead load−weight of haunch (klf)

w_s = uniform dead load−weight of deck slab (klf)

V_c = nominal shear resistance provided by tensile stresses in the concrete (kip)

V_n = nominal shear resistance of the section considered (kip)

V_{ni} = nominal interface shear resistance (kip)

V_{ri} = factored interface shear resistance (kip)

V_p = component in the direction of the applied shear of the effective prestressing force; positive if resisting the applied shear (kip)

V_s = shear resistance provided by the transverse reinforcement at the section under investigation as given by AASHTO 5.8.3.3-4, except V_s shall not be less than V_u/ϕ (kip)

V_u = factored shear force (kip)

v_u = average factored shear stress on the concrete (ksi)

V_{ui} = factored interface shear resistance (kip)

x = distance from left end of girder (ft)

Y = distance from the neutral axis to a point on individual component (in.)

y_b = distance from the neutral axis to the extreme bottom fiber of PC girder (in.)

Y_{BC} = distance from the centroid to extreme bottom fiber of composite section (in.)

Y_{BCt} = distance from the neutral axis to the bottom extreme fiber of the composite, at service, transformed (in.)

Y_{Btf} = distance from the neutral axis to bottom extreme fiber of PC girder, at service, transformed (in.)

Y_{Bti} = distance from the neutral axis to bottom extreme fiber of PC girder, at initial, transformed (in.)

y_{bts} = centroid of all tensile reinforcement (in.)

y_i = distance from centroid of section i to centroid of composite section (in.)

y_t = distance from the neutral axis to the extreme top fiber of PC girder (in.)

Y_{TC} = distance from the centroid to extreme top fiber of composite section (in.)

Y_{tg} = distance from centroid of the composite section to the extreme top fiber of the

PC girder (in.)

Y_{TGCt} = distance from centroid of the composite section to the extreme top fiber of the PC girder (in.)

Y_{Tti} = distance from neutral axis to the extreme top fiber of the PC girder, transformed (in.)

α = angle of inclination of transverse reinforcement to longitudinal axis (°) and total angular change of prestressing steel path from jacking end to a point under investigation (rad)

β = factor relating effect of longitudinal strain on the shear capacity of concrete, as indicated by the ability of diagonally cracked concrete to transmit tension (unitless)

β_1 = ratio of the depth of the equivalent uniformly stressed compression zone assumed in the strength limit state to the depth of the actual compression zone

Δ_{br} = deflection due to barrier weight (in.)

Δ_g = camber at midspan at erection due to girder self-weight (in.)

$\Delta_{g,erect}$ = camber at midspan at erection due to long-term effects of prestressing force and girder self-weight (in.)

Δ_{ES} = change in length due to elastic shortening

Δf_{pES} = sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads (ksi)

Δf_{pLT} = losses due to long-term shrinkage and creep of concrete and relaxation of prestressing steel (ksi)

Δf_{pR} = an estimation of relaxation loss taken as 2.4 ksi for low relaxation strand, 10 ksi for stress relieved strand, and in accordance with manufacturers recommendation for other types of strand (ksi)

Δf_{pT} = total change in stress due to losses (ksi)

Δ_{fw} = deflection due to future wearing surface (in.)

Δ_p = camber at midspan due to prestressing force at release (in.)

Δ_s = instantaneous deflection due to weight of deck slab (in.)

ε_{cu} = failure strain of concrete in compression (in./in.)

ε_t = net tensile strain in extreme tension steel at nominal resistance (in./in.)

ε_x = longitudinal strain in the web reinforcement on the flexural tension side of the member (in./in.)

θ = angle of inclination of diagonal compressive stresses

γ_1 = flexural cracking variability factor

γ_2 = prestress variability factor

γ_3 = ratio of specified minimum yield strength to ultimate tensile strength of reinforcement

γ_h = correction factor for relative humidity of ambient air.

γ_{st} = correction factor for specified concrete strength time at of prestress transfer to concrete member

ϕ = resistance factor

μ = coefficient of friction (unitless)

ω = angle of harped strands

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