CHAPTER 5.4 PRECAST PRETENSIONED BOX GIRDERS

TABLE OF CONTENTS

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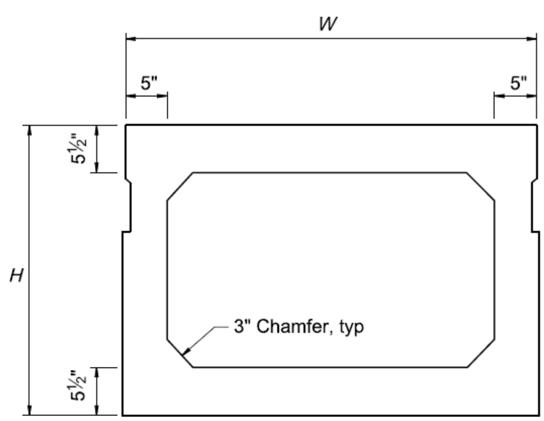
5.4.1 INTRODUCTION

Precast, pretensioned concrete box girders (precast box girders) are typically used for relatively short span structures with a limited or inadequate temporary clearance for falsework, as discussed in Section 5.3.1. Placed side-by-side, the top flange can serve as the driving surface, which makes them ideal for Accelerated Bridge Construction (ABC) applications. However, an overlay consisting of reinforced concrete or polyester concrete is needed to adjust the profile grade and achieve a continuous cross slope. For detailed discussions, references are made to Naaman (2004) and Snyder (2010).

5.4.2 TYPICAL SECTIONS AND SPAN LENGTHS

Typical girder sections should be based on the American Association of State Highway and Transportation Officials/Precast-Prestressed Concrete Institute (AASHTO/PCI) Sections, as described in the Precast Concrete Institute (PCI) *Bridge Design Manual* (PCI, 2014). Four sections with designations of BI through BIV represent depths ranging from 27 to 42 inches, respectively. The section dimensions, which are illustrated in Figure 5.4-1, can be adjusted for non-standard and variable exterior and interior dimensions without a significant cost increase if the strands are straight and the width is less than 48 inches (PCI, 2014). Since prestressing strands are straight, stresses can be controlled by partial debonding, as discussed in Section 5.3.2.3. Figure 5.4-2 shows a box girder with void forms, prestressing stands and reinforcing in precast yard before closing the forms.



Dimensions (inches)

Type	*W*	*H*
BI-36	36	27
BII-36	36	33
BIII-36	36	39
BIV-36	36	42
BI-48	48	27
BII-48	48	33
BIII-48	48	39
BIV-48	48	42

Figure 5.4-1 Precast Box Girders (PCI, 2014)

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Figure 5.4-2 Precast Box Girder Void Forms, Strand, and Reinforcing Steel (Courtesy of TYLin)

For preliminary design, the girder type and associated depth can be taken from Table 5.4-1. This table is based on *PCI Bridge Design Manual*, Chart BB-2 (PCI, 2014), and preliminary calculations. The girder depth should be verified to meet all strength and service limit states including deflection requirements. The minimum specified depths for precast box girder bridges, including a cast-in-place (CIP) concrete deck, is 0.030*L* and 0.025*L* for simply supported and continuous spans, respectively, where L is the span length (AASHTO Article 2.5.2.6.3). Predicting deflections of precast box girders is challenging, and the issue is magnified with longer spans. Therefore, coordination with the Precast Concrete Specialist is recommended for spans greater than 100 feet.

Precast box girders are typically placed so the soffit matches the cross-slope and the average profile grade of the span. Since the sections are wide relative to the depth, the girders are stable at cross slopes up to ten percent, which is the upper limit specified in the *Highway Design Manual* Section 208.2 (Caltrans, 2020). The girder ends can be supported with thin neoprene strips on a non-level bent cap and integral abutment seats. Level bearings designed are recommended for girder ends on seat abutments. Greased bearings are not recommended.

During typical fabrication and subsequent curing operations, the girders will deflect upward vertically (camber) and sweep horizontally, as shown in Figure 5.4-3 These deflections are typically not uniform for all girders within a span and must be accounted for in the design.

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Table 5.4-1 Recommended Span Lengths for Preliminary Design

Type	Depth (in)	Span (ft)**	
		No Concrete Deck	With Concrete Deck*
BI	27	75	90
BII	33	90	100
BIII	39	110	110
BIV	42	115	115

*Based on a 6-inch-deep CIP concrete deck

**Coordination with the Precast Concrete Committee Specialist is recommended for spans greater than 100 feet.

Sweep is caused by form and prestressing strand misalignment, strand tensioning variability, thermal effects due to sun exposure on one side, and improper storage at the fabrication site. Since differential sweep could result in an imperfect fit, a small gap should be specified between girders. This gap should be at least 1.5 times the sweep allowed in Section 10.11 of the *Tolerance Manual for Precast and Prestressed Concrete Construction, MNL-135-00* (PCI, 2000), which is referenced in *Standard Specifications* Section 90-4.03 (Caltrans, 2018).

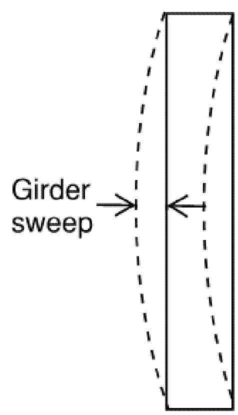

Figure 5.4-3 Girder Plan View Schematic Showing Horizontal (Sweep) Deflection

Camber is a result of eccentrically applied prestresses in the girder, as discussed in Section 5.4.4. Differential camber could result in a non-uniform cross slope and misalignment of the prestress tendons in the transverse diaphragms, as discussed in Section 5.4.5.

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5.4.3 LONGITUDINAL JOINTS

The longitudinal joints between girders should have a keyway filled with non-shrink Portland cement grout or a closure with Ultra-High Performance Concrete (UHPC) to create a water barrier, which should be designed to resist relative deflection of the precast units under applied loads, as discussed in Section 5.4.5. Alternatively, a cast-in-place concrete deck can provide similar water protection and relative-deflection resistance, as discussed in Section 5.4.4.

For transversely post-tensioned box girders with longitudinal joints filled with non-shrink grout, AASHTO Article 5.12.2.3.3c requires the prestress after losses not to be less than 0.25 ksi with a compressed depth of the joint not being less than 7.0 inches. The intent of this requirement is to (1) minimize tensile stresses that cause longitudinal cracking at the joints to prevent water penetration, (2) transfer shears across the joint to distribute live loads to multiple girders, and (3) minimize differential deflections between girders and prevent reflective cracks in overlays.

Longitudinal joint details shown in Figure 5.4-4 include a 12-inch-deep keyway filled with non-shrink grout, which is like the detail in the *PCI Bridge Design Manual* (PCI, 2014). A foam seal is needed to prevent grout from leaking during placement. Increasing the keyway depths will not significantly improve performance and reduce the prestress acting on the transverse diaphragms. Note that UHPC joints with relatively short reinforcement extensions into the keyway can transfer shear and prevent relative deflection without transverse post-tensioning, as discussed in Section 5.4.5.

A gap should be provided to account for sweep and side form imperfections. The size of the gap can vary between fabrication plants, and the design should have the flexibility to accommodate variable gaps. Sweep and side form imperfections may result in unanticipated contact points. These contact points at the bottom of the girder could spall during transverse prestressing operations. To reduce the risk of spalling, a notch, as shown in Figure 5.4-4 at the bottom of the girders is recommended (PCI, 2014).

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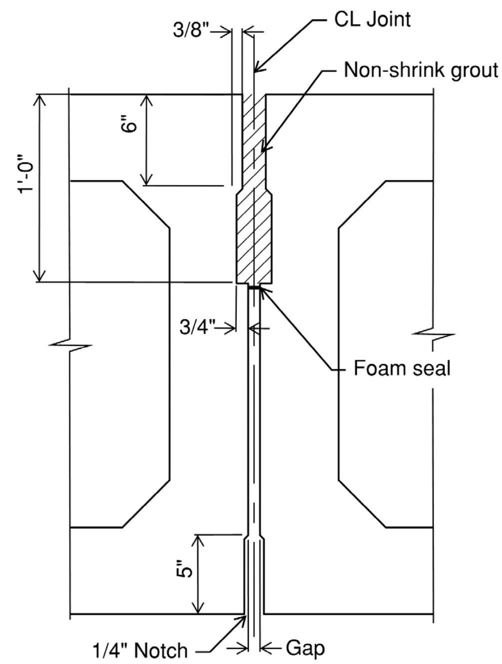

Figure 5.4-4 Longitudinal Joint Details

5.4.4 OVERLAYS AND CAST-IN-PLACE CONCRETE DECKS

Precast box girders are cast on a flat surface. Transfer of the prestressing strand to the concrete causes the girder to deflect upward at midspan. This deflection continues to increase until the placement of a concrete deck, barriers, sidewalks, and overlays. Further, this deflection can be variable resulting in minor elevation differences between girders. Consequently, the desired driving surface profile may not be achievable without an overlay or deck grinding. Overlays may consist of the following:

Polyester Concrete: Although a polyester concrete overlay does not provide structural resistance, it is durable, waterproof, and provides a reliable driving surface within hours of application that make it ideal for ABC. With a significant tensile capacity and relatively low elastic modulus, polyester concrete overlays stretch without cracking, thus improving the long-term performance of the bridge deck.

A minimum overlay thickness between 1.0 and 3.0 inches is recommended. Since the polyester concrete requires specialized equipment and personnel to produce and place, the cost can be significant. Further, increasing the thickness of the polyester concrete overlays will not necessarily improve the performance. Consequently, the thickness of polyester concrete overlay should be minimized. However, overlays thicker than three inches could be warranted for the following reasons:

- Achieving the desired profile grade requires variation in thickness of the overlay unless a vertical curve is specified that matches the camber of the girder after erection.

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- Camber variation between girders can be significant. Therefore, the overlay thickness should be large enough to accommodate this variability to create a constant cross slope. Alternatively, the deck surface could be ground prior to applying the overlay to reduce the overlay thickness.

Polyester concrete requires a dry concrete surface that has been fully cured to develop a complete bond. Note that the *Standard Specifications* Section 60-3.04B(1)(d) (Caltrans, 2018) requires a 28-day concrete cure duration before applying polyester concrete overlay to Portland cement concrete.

Cast-in-Place (CIP) Concrete Deck (Structural Overlay): A CIP concrete deck reinforced with an orthogonal grid of reinforcement and connected to the girder with stirrups:

- Forms a composite section with the girder thereby increasing flexure and shear resistance with the interface shear connection provided by the girder stirrups;
- Acts as a water-resistant surface and allows for grading of the profile surface; and
- Provides transverse continuity, which eliminates the need for transverse post-tensioning and intermediate diaphragms.

To achieve transverse continuity, AASHTO Article 5.12.2.3.3f requires that “the thickness of structural concrete overlay shall not be less than 4.5 in. An isotropic layer of reinforcement shall be provided in accordance with the requirements of AASHTO Article 5.10.6. The top surface of the precast components shall be roughened.” It should be noted that AASHTO Article 5.10.6 specifies the minimum reinforcement for shrinkage and temperature.

The *PCI Bridge Design Manual* Section 8.9.1.2 (PCI, 2014) recommends a minimum concrete deck thickness of 5.9 inches. Note that this minimum thickness is typically at midspan and increases toward the supports due to camber, as discussed previously.

For multiple span bridges, the concrete deck can be made continuous for live and superimposed dead loads with the addition of longitudinal reinforcement over the bent cap.

5.4.5 TRANSVERSE CONTINUITY

Precast box girder bridges can be constructed by placing the girders side-by-side to create a roadway surface without constructing a deck. Without an adequate transverse connection, these girders will not deflect equally under live loads. This unequal differential displacement could result in reflective cracking in the overlay, and the intrusion of water into the joint between the girders. These cracks should be prevented because water penetration between the girders could result in staining and corrosion if subjected to a marine environment or deicing chemicals. Further, limiting differential deflection distributes loads to adjacent girders, which improves efficiency.

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There are several approaches to limit this relative displacement to create transverse continuity including transverse post-tensioned diaphragms, CIP concrete decks, grouted longitudinal joints or keyways, and UHPC closures.

Transverse post-tensioned diaphragms: Transverse continuity can be achieved in precast box girders with transverse post-tensioned diaphragms to reduce the girder relative displacement to an acceptable amount. Note that post-tensioned diaphragms are not needed for bridge spans with a concrete deck because the deck resists relative deflection and prevents water penetration.

Section 8.9.3.1 of the *PCI Bridge Design Manual*, (PCI 2014) recommends diaphragm layouts listed in Table 5.4-2 for the preliminary design. Due to the possibility of cracking at the joints and the resulting loss of stiffness, the use of non-post-tensioned rods is not recommended.

Table 5.4-2 Recommended Diaphragm Locations for Transverse Continuity (PCI, 2014)

Span Length (ft)	No.	Locations
$L \leq 60$	3	Ends and at midspan
$60 < L \leq 100$	5	Ends and at quarter points in the span
$L > 100$	> 5	Space at 25 feet

Since transverse prestress tendon lengths are relatively short, anchor set losses can be significant. Therefore, post-tensioning rods, per ASTM A722, Type II, are recommended because the anchor set can be as low as 0.0625 inches as compared to 0.375 inches for strand anchorages (AASHTO, C5.9.3.2.1).

Post-tensioning ducts are spliced within the grouted blockouts between the girders, as shown in Figure 5.4-5. Since the blockouts are filled with grout between the girders prior to post-tensioning, ducts shall be water-tight at the coupler. Caltrans Standard Specifications require testing to demonstrate that the ducts can contain an air pressure of 50 psi with minimal leakage prior to grouting. Meeting this criterion can be challenging, as duct splices are to be made within a relatively narrow space, and differential cambers between girders creates minor misalignments of the duct that must be reconciled over a relatively short distance. Therefore, the blockouts should have a minimum width of 3.0 inches measured parallel to the transverse diaphragm to allow an adequate room for duct splicing and waterproofing, as shown in Figure 5.4-5.

The transverse prestress can be designed based on the post-tensioning design chart developed by Hanna et. al (2007) and the design procedure developed by El-Remaily, et. al (1996) in Section 8.9.3.5 of the *PCI Bridge Design Manual* (PCI, 2014). This method provides recommendations for the prestress force required to keep differential deflections within an acceptable limit of 0.02 inches. The required prestress force should compress the full-depth diaphragm sufficiently to remain in compression and uncracked under service loads.

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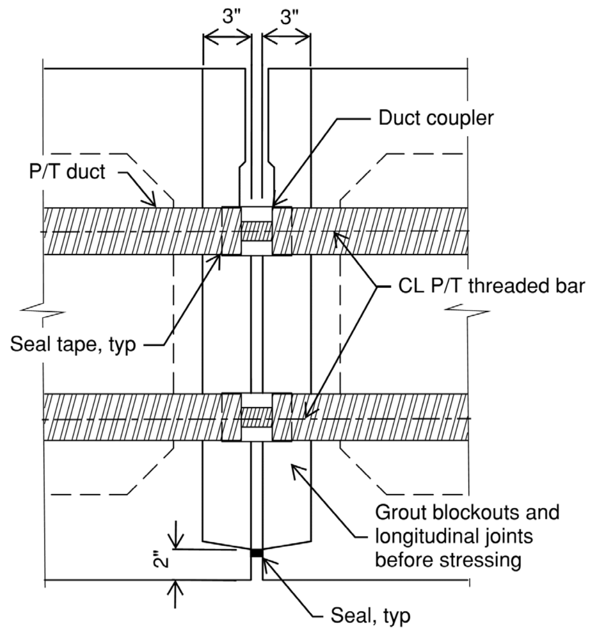

Figure 5.4-5 Blockouts in Transverse Post-Tensioned Diaphragms

Based on the methods and results of El Remaily, et al (1996), Hanna, et al (2007) developed an equation for determining the amount of prestress (P) per foot of the bridge length of adjacent girder bridge spans, as follows:

$$P = \left(\frac{0.9W}{D} - 1.0\right) K_L K_S \leq \left(\frac{0.2W}{D} + 8.0\right) K_L K_S \qquad \text{(AASHTO 5.4.5-1)}$$

where:

P = transverse post-tensioning force required

D = superstructure depth

W = bridge width

K_L and K_S are correction factors for span-to-depth ratios and skew, respectively.

$$K_L = 1.0 + 0.003\left(\frac{L}{D} - 30\right) \qquad \text{(AASHTO 5.4.5-2)}$$

where:

$$K_S = 1.0 + 0.002\theta$$

L = span length

θ = skew angle

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The inequality of Equation 5.4.5-1 is intended to limit the prestressing force for wide bridges (exceeding 52 feet), where positive bending moments in the diaphragm control the design. Note this equation has been adopted in the *PCI Bridge Design Manual* (PCI, 2014). Figure 5.4-6 shows the transverse diaphragm blockout.

Figure 5.4-6 Transverse Diaphragm Blockout, Courtesy of Confab, CA

UHPC Closures: The transverse continuity can be achieved with UHPC within a longitudinal closure. This closure includes reinforcement that extends from the girder (FHWA, 2014). The steel fibers in UHPC provide the confinement resulting in a short embedment length of the rebar extending from the girders into the closures. For UHPC closures, an embedment length of eight bar diameters ($8d_b$) is sufficient for most applications including epoxy coated reinforcement (FHWA, 2014).

An example detail of a UHPC closure is shown in Figure 5.4-7, where the transverse reinforcement extends beyond the vertical face. These bars must be installed as bent bars to fit within the vertical side forms and will be straightened after the girder is cast. Alternatively, form-saver mechanical couplings could be used, where the embedded bars are installed after the forms are released.

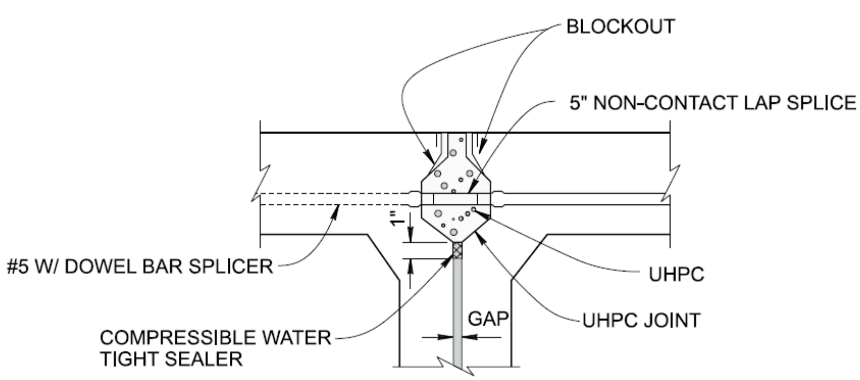

Figure 5.4-7 Precast Box Girder UHPC Closure Detail

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The amount of the transverse reinforcement embedded in the UHPC joint could be estimated using Equation 5.4.5-1 and verified through grillage analysis models. The connection between girders should effectively perform as a flexural pin, and transverse members should be modeled as girder elements connected with moment-released nodes at the longitudinal joint. The shear resistance should be analyzed at the girder joint interface using the interface shear friction.

5.4.6 DESIGN FLOW CHART

The following chart (Figure 5.4-8) shows the typical steps for designing a single-span precast box girder bridge connected with the transverse post-tensioning.

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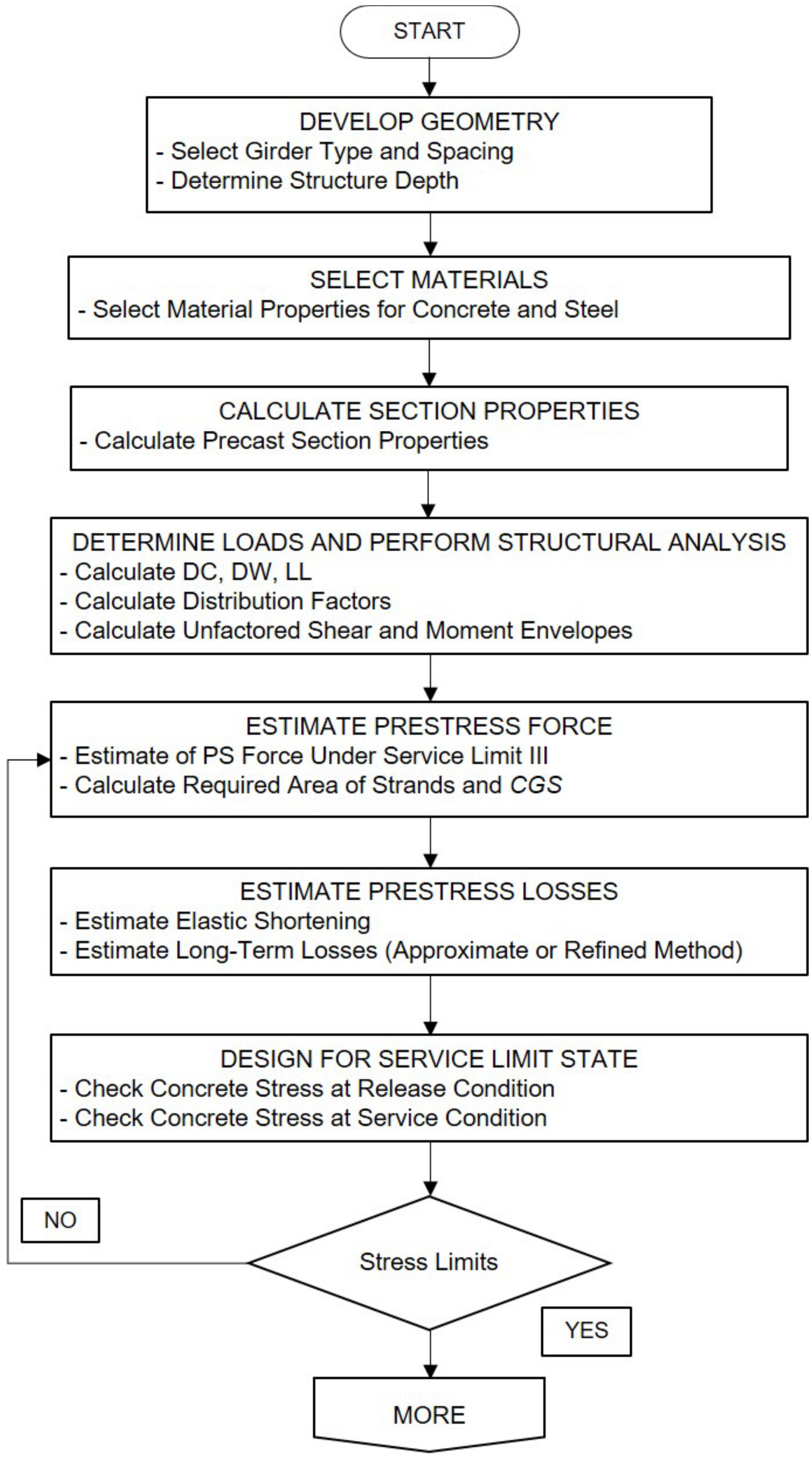

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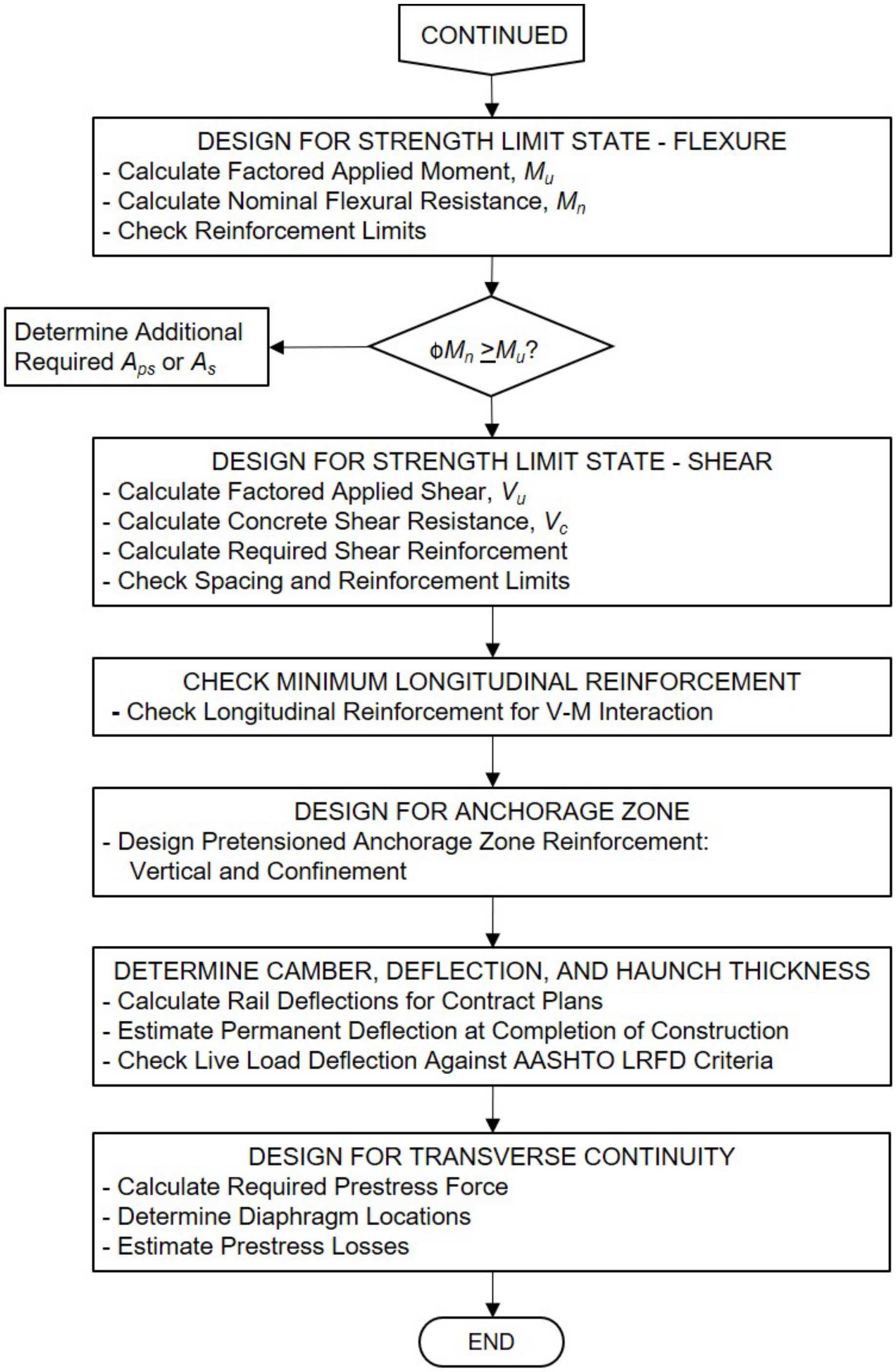

Figure 5.4-8 Precast/Prestressed Concrete Box Girder Design Flow Chart with Transverse Post-Tensioning

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5.4.7 DESIGN EXAMPLE

This example illustrates the design procedures for a typical precast box girder bridge designed in accordance with AASHTO-CA BDS-8 (AASHTO, 2017; Caltrans, 2019).

To demonstrate the process, a three-span bridge with a 10-degree skew angle is designed using precast box girders with a CIP concrete deck. The HL93 vehicular live load is used for Service I, Service III, and Strength I limit state design in accordance with AASHTO Article 3.6.1.2. The P15 permit design truck is used for Strength II limit state design in accordance with Caltrans Amendment Article 3.6.1.8 (Caltrans, 2019). Concrete stress checks for initial and final service limit states are based on the transformed sections. The approximate method of estimating time-dependent losses, per AASHTO Article 5.9.3.3) is used to calculate prestress losses. The shear design is based on the sectional design model of AASHTO Article 5.7.3.

Major design steps include: (1) establishing structural geometry, (2) selecting girder type, (3) selecting materials, (4) performing structural analysis, (5) estimating prestress force, (6) estimating prestress losses, (7) service limit state design, (8) strength limit state design for flexure and shear, (9) anchorage zone design, (10) determining girder deflections, and (11) determining minimum deck thickness at the supports.

5.4.7.1 Problem Statement

A 252.5-foot long three-span overhead bridge is proposed to carry pedestrian and vehicular traffic over railway and roadway improvements. To meet the vertical clearance requirement a precast box girder bridge is recommended. Figures 5.4-9 and 5.4-10 show the elevation and plan views of the bridge. The effective span lengths measured from the centerline of the bearing to the centerline of the bearing are 75-feet, 97-feet, and 75-feet, for Spans 1 through 3, respectively. The precast box girders are placed next to each other with a CIP concrete deck to provide continuity both transversely between girders and longitudinally over the bent caps.

The 60-foot-wide bridge carries three 12-foot-wide traffic lanes, a 5-foot wide, an 8-ft wide shoulder, and a 6.5-foot-wide sidewalk. A Type 836 barrier separates the roadway and the sidewalk. A 1.0-foot-wide area is provided for anchoring the metal fence as shown in Figure 5.4-11. The riding surface is comprised of a 6.0-inch-thick CIP concrete deck.

It is required to design a typical interior box girder of Span 2 in accordance with AASHTO-CA BDS-8 for all limit states.

Exterior girders should be designed for both exterior and interior configurations to allow for future widening and are not illustrated in this example.

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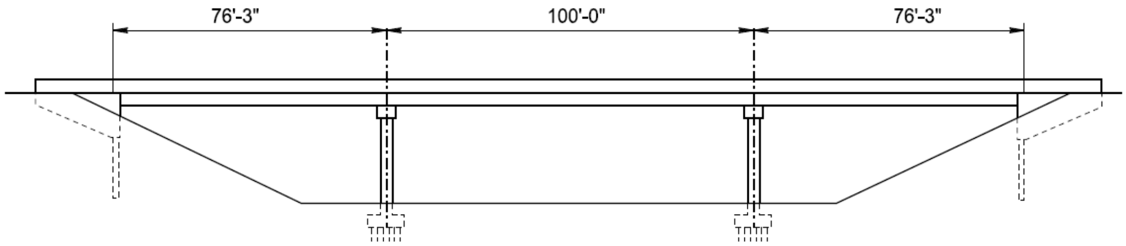

Figure 5.4-9 Elevation View of the Example Bridge

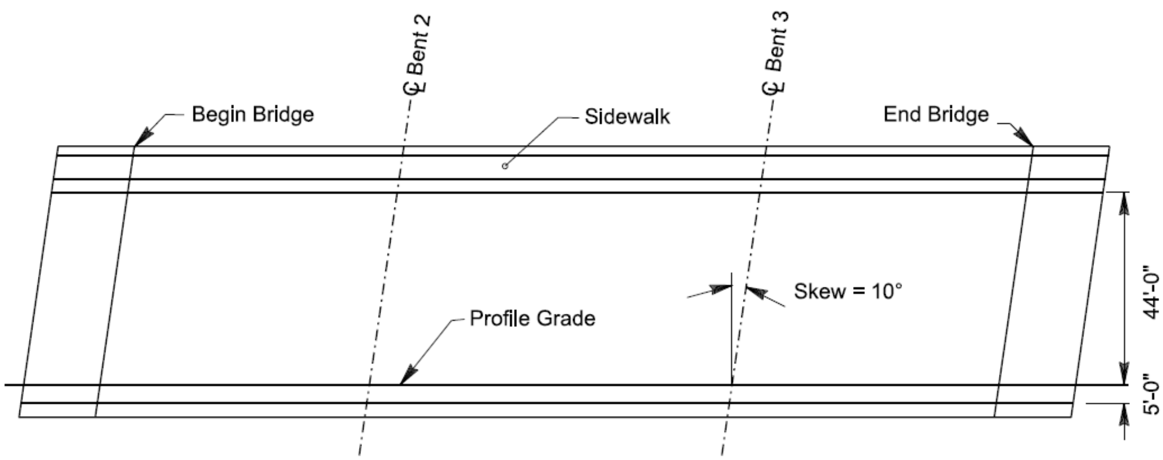

Figure 5.4-10 Plan View of the Example Bridge

5.4.7.2 Select Girder Depth, Type, and Number of Girders

For a 100 ft span length, a precast box girder section is ideal for overhead bridge structures with limited vertical clearance. The minimum structure depth-to-span-length ratio (*D*/*L*) in AASHTO Table 2.5.2.6.3-1 is 0.025 for continuous precast box girder bridges.

$$\text{Span length, } L = 100 \text{ ft (composite continuous structure)}$$
$$= 97 \text{ ft (bearing to bearing)}$$

Assuming: $$\frac{\text{Structure Depth, } D_s}{\text{Span Length, } L} \geq 0.025$$

$$D_s \geq 0.025\left(100\right) \geq 2.5 \text{ ft}$$

Target span ranges for Types BI through BIV, with depths ranging from 27 to 42 inches, are listed in Table 5.4-1. For a span of 100 feet, the BII section having a depth of 33 inches with a 6-inch deep concrete deck has the closest target span and meets the recommended minimum depth requirement.

The interior girders will be 48 inches wide, and the exterior girders will have a reduced

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width.

The typical section should be detailed with a gap between girders to accommodate differential sweep, as discussed in Section 5.4.2. Per Section 10.11 of the *Tolerance Manual for Precast Prestressed Concrete* (PCI, 2000), the allowed sweep is 0.5 inches for spans greater than 60 feet. Therefore, according to Section 5.4.2:

$$\text{Gap} \geq 1.5\ x\ 0.5 \text{ in} = 0.75 \text{ in}$$

$$\text{Number of girders} = \frac{60' \text{bridge width}}{4' \text{ girder width}} = 15 \text{ girders}$$

$$\text{Total width of gaps} = 14 \text{ gaps x } 0.75 \text{ in} = 10.5 \text{ in} = 0.88 \text{ ft}$$

Use 13 - 48 inch (4.00 ft) wide interior girders and 2 - 42.75 inch (3.56 ft) wide exterior girders.

Total bridge width, $W = 13(4.00) + 2(3.56) + 0.88 = 60.0$ ft

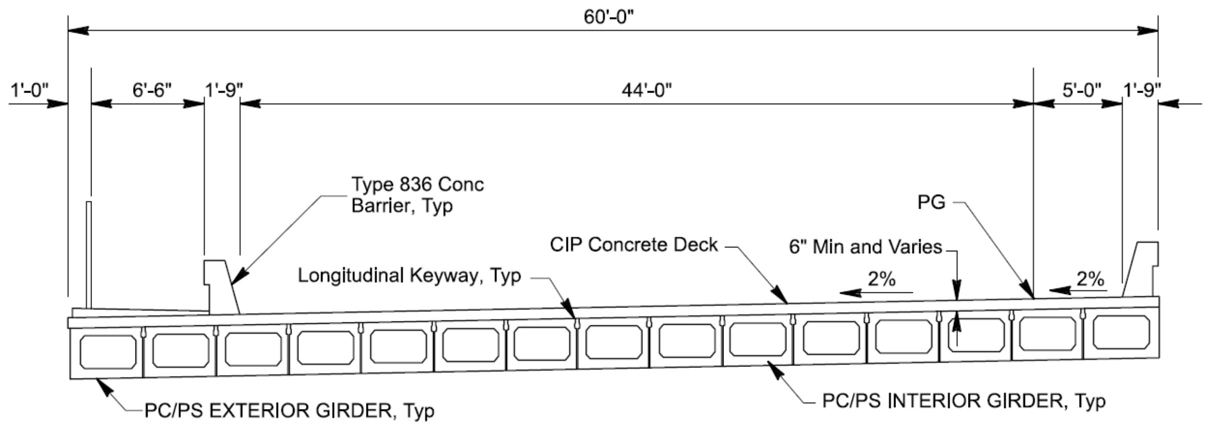

Figure 5.4-11 Example Bridge Typical Cross Section

5.4.7.3 Establish Loading Sequence

The loading sequence and corresponding stresses for a three-span precast box girder bridge are normally considered at three distinct stages, as summarized in Table 5.4-3. All loading is resisted initially by the girders alone until the structural concrete deck has cured. After that, the composite section resists live loads and superimposed dead loads.

Note: Per Caltrans practice, transportation (shipping, handling, and erection) is generally the responsibility of the contractor and precast concrete fabricator. Note that construction loads shall be considered in the design of precast elements.

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Table 5.4-3 Typical Stages of Loading for Three-Span Precast Box Girder Bridge

Stage	Location	Construction Activity	Loads	Load Resistance
I	Casting Yard	Cast and Stress girder (Transfer)	DC (Girder)	Girder
IIA	On Site	Erect girder, Cast Deck Slab	DC (Girder and wet deck), Construction Loads	Girder
IIB	Final Location	Construct Barrier Rails and Sidewalk	DC (Girder, Deck, and Barrier Rails), DW (future wearing and utilities)	Girder/Deck composite section
III	Final Location	Open to Traffic	DC (Girder, Deck, and Barrier Rails) DW (future wearing and utilities) LL (Vehicular Loading, HL93 or P15)	Girder/Deck composite section

5.4.7.4 Select Materials

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

- Concrete compressive strength and modulus of elasticity

E_c = modulus of Elasticity

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

w_c = unit weight of concrete for the purpose of calculating the elastic modulus. Note the unit weight for calculating loads is higher to account for the weight of strands, reinforcing steel, and forms that remain inside the section.

K_1 = correction factor = 1.0

At transfer, the required concrete compressive strength (f'_{ci}) can significantly affect the cost, as fabricators rely on the daily use of prestressing beds. Therefore, f'_{ci} should be kept to a minimum, while staying within allowable temporary stresses (PCI, 2014). Caltrans requires f'_{ci} to be a minimum of 4.0 ksi for pretensioned members.

$$f'_{ci} = 4.0 \text{ ksi}$$

$$w_c = 0.145 \text{ ksi for } f'_c \leq 5.0 \text{ ksi} \qquad \text{(AASHTO Table 3.5.1-1)}$$

$$E_{ci} = 120{,}000 \; x \; 1.0 \; x \; 0.145^{2.0} \; x \; 4.0^{0.33} = 3{,}987 \text{ ksi}$$

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At 28 days, the required concrete compressive strengths (f'_c) of up to 6.0 ksi are readily attainable. Conservatively,

$$f'_c = 5.0 \text{ ksi}$$

$$E_c = 120{,}000 \; x \; 1.0 \; x \; 0.145^{2.0} \; x \; 5.0^{0.33} = 4{,}291 \text{ ksi}$$

CIP Concrete Deck

$$f'_{ci} = 4.0 \text{ ksi} \qquad \text{(AASHTO 5.4.2.1)}$$

$$E_{ci} = 120{,}000 \; x \; 1.0 \; x \; 0.145^{2.0} \; x \; 4.0^{0.33} = 3{,}987 \text{ ksi}$$

- Prestressing steel:

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

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

f_{pu} = nominal tensile strength of Grade 270 strand = 270 ksi (AASHTO Table 5.4.4.1-1)

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

f_{pj} = Initial jacking stress = 0.75 f_{pu} = 202.5 ksi

(CA Table 5.9.2.2-1, 2019)

E_p = modulus of elasticity of prestressing steel = 28,500 ksi

(AASHTO Article 5.4.4.2)

- Mild steel – A706 reinforcing steel:

f_y = Nominal yield strength, = 60 ksi

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

5.4.7.5 Calculate Gross Section Properties

The gross section properties for a typical interior girder are used to calculate dead loads and deflections, and the transformed section properties are used to check the concrete stress limits at each stage. Section property calculations are based on a girder width of 48 inches, but the spacing is 48.75 to account for the gap between girders. Figure 5.4-12 shows the cross section of the girder. The keyway effects on section properties are ignored.

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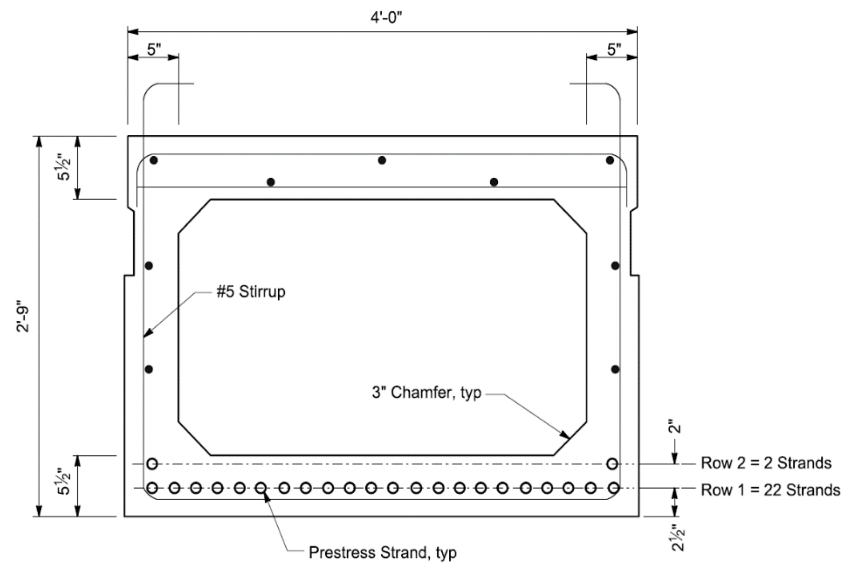

Figure 5.4-12 Precast Box Girder Cross Section

The area and the moment of inertia of the rectangle forming the outer perimeter of the precast box girder are as follows:

$$A_{rect} = 48 \ x \ 33 = 1{,}584 \text{ in.}^2$$

I_{rect} = moment of inertia of the rectangle

$$I_{rect} = \frac{48(33)^3}{12} = 143{,}748 \text{ in.}^4$$

The area and the moment of inertia of the void forming the inner dimensions of the box girder are as follows:

$$A_{void} = 38 \ x \ 22 = 836 \text{ in.}^2$$

$$I_{void} = \frac{38(22)^3}{12} = 33{,}719 \text{ in.}^4$$

The area of the chamfers at the void corners (total 4)

$$A_{cham} = 0.5 \ x \ 3^2 x \ 4 = 18 \text{ in.}^2$$

The vertical distance from the centroid of the chamfer to the centroid of the girder

$$d_c = 16.5 - 5.5 - 3/3 = 10 \text{ in.}$$

The approximate moment of inertia of the chamfers measured to the centroidal axis of the girder

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$$I_{cham} = A_{cham} x d_c^2 = 18 \ x \ 10^2 = 1{,}800 \text{ in.}^4$$

The area of the precast box girder (gross)

$$A_g = A_{rect} - A_{void} + A_{cham} = 1{,}584 - 836 + 18 = 766 \text{ in.}^2$$

The moment of inertia of the precast box girder (gross)

$$I_g = I_{rect} - I_{void} + I_{cham} = 143{,}748 - 33{,}719 + 1{,}800 = 111{,}829 \text{ in.}^4$$

The section modulus with the extreme bottom fiber in tension (gross)

$$S_b = \frac{I_g}{y_b} = \frac{111{,}829}{16.5} = 6{,}778 \text{ in.}^3$$

5.4.7.6 Determine Loads

5.4.7.6.1 Dead Load

Loads on the non-composite section include the girder self-weight and the wet concrete weight of the CIP concrete deck. The unit weight (w_c) of 0.15 kips per cubic foot (kcf) includes the weight of concrete, prestressing strand, rebar, and formwork.

Self-weight of the concrete girder

$$w_g = A_g y_c = \frac{766}{144}(0.15) = 0.798 \text{ kip/ft}$$

Weight of the CIP concrete deck with a thickness of 6.0-inches and a girder spacing of 48.75 inches

$$w_s = \frac{6(48.75)}{144}(0.15) = 0.\ 305 \text{ kip/ft}$$

Total dead load resisted by the girder

$$DC1 = w_g + w_s = 0.798 + 0.305 = 1.103 \text{ kip/ft}$$

Dead loads acting on the composite section, include the weight of the barriers, sidewalks, a future wearing surface, and utilities. (AASHTO Article 4.6.2.2.1) states that permanent loads may be distributed uniformly among the girders if the following conditions are met:

- Width of the deck is constant. (OK)
- Number of girders, N_b, is not less than four (N_b = 15). (OK)
- Girders are parallel and have approximately the same stiffness. (OK)
- The roadway part of the overhang, $d_e \leq 3.0$ ft (d_e = 0.0). (OK)

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- Curvature in the plan is less than specified in the LRFD Specifications (curvature = 0.0º). (OK)
- Cross-section is consistent with one of the cross-sections shown in AASHTO-CA BDS Table 4.6.2.2.1-1 (Type (g)). (OK)

Since these criteria are satisfied, the permanent loads applied to the composite section are evenly distributed across the width based on the dead load distribution factor (DF_{DL}), which can be determined as:

$$DF_{DL} = \frac{\text{Tributary Width}}{\text{Bridge Width}} = \frac{48.75}{60 \times 12} = 0.0677$$

Note that the tributary width includes the girder width and the 0.75-inch gap calculated in Section 5 4.7.2.

Weight of the Type 836 barrier rail on both sides of the deck (concrete area = 479.5 in.^{2} each) distributed to each girder using the DF_{DL}:

$$w_{br} = \frac{2(479.5)}{144}(0.15)(0.0667) = 0.068 \text{ kip/ft}$$

The sidewalk is 7.5 feet wide with a thickness that varies from 3 inches at the face of the barrier to 6 inches at the edge of the deck and has a concrete area = 405 in.^{2}. The weight of the sidewalk is as follows:

$$w_{sw} = \frac{(405)}{144}(0.15)\ (0.0677) = 0.029 \text{ kip/ft}$$

Permanent loads acting on the composite section:

$$DC2 = w_{br} + w_{sw} = 0.068 + 0.029 = 0.096 \text{ kip/ft}$$

A future wearing surface of 0.035 ksf loading the roadway driving surface measuring 49 feet in width, has the following load effect:

$$DW = 49(0.035)(0.0677) = 0.116 \text{ kip/ft}$$

5.4.7.6.2 Live Load

The bridge is designed for the strength I - HL93 vehicular live load and the strength II - P15 design permit truck as described in Section 5.4.7. The live load effects are calculated by the bridge analysis software.

5.4.7.7 Perform Structural Analysis

5.4.7.7.1 Unfactored Bending Moments and Shear Forces due to DC and DW

Dead load moments for an interior girder of Span 2 are obtained from two separate models that include (1) a simply supported non-composite girder spanning between

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supports after the erection, which includes the weight of the girder and the wet concrete of the CIP concrete deck, and (2) the dead and live load acting on a continuous three span composite structure. Unfactored shears and moments are summarized in Table 5.4-4.

Table 5.4-4 Unfactored Shear Forces and Bending Moments due to *DC* and *DW*

Location		Non-composite (*DC1*)		Composite (*DC2*)		Wearing surface (*DW*)	
Dist/Span (*X/L*)	Location (ft)	Moment (kip-ft)	Shear (kip)	Moment (kip-ft)	Shear (kip)	Moment (kip-ft)	Shear (kip)
Bearing	0	0	53.5	-67.6	4.6	-81.6	5.5
H/2*	1.63	85.7	51.7	-60.2	4.4	-72.6	5.4
Transfer**	2.5	155.5	50.2	-56.3	4.4	-67.9	5.3
0.1 L	9.7	449.6	43.2	-29.2	3.7	-35.2	4.5
0.2 L	19.4	820.3	32.4	2.6	2.8	3.2	3.4
0.3 L	29.1	1,085	21.6	25.4	1.9	30.6	2.2
0.4 L	38.8	1,244	10.8	39.0	0.9	47.1	1.1
0.5 L	48.5	1,297	0	43.6	0	52.6	0

*Assumed location of maximum shear.

**Prestress transfer length ($60d_{bstrand}$) from the end of the girder.

For Span 2, the girders have an effective length of 97 feet measured from the centerline of the bearing to the centerline of the bearing for the self-weight and the weight of the cast-in-place deck. The precast box girder is assumed to be pinned at the bents with rollers at the abutments under composite conditions. For superimposed dead loads and live loads, Span 2 is continuous with the adjacent spans with an effective length of 100 feet.

5.4.7.7.2 Unfactored Bending Moments and Shears due to Live Loads

The live load moments and shears distributed to the individual girders are calculated by using the simplified distribution factor formulas in Articles 4.6.2.2.2 and 4.6.2.2.3, respectively. As shown previously, the conditions of Article 4.6.2.2.1 are satisfied for this example bridge. Therefore, the simplified distribution factor formulas are applied to the interior girder design in the following sections.

Live Load Moment Distribution Factor (Interior Girders)

The live load distribution factor for moments (*DFM*, lanes/girder), for an interior girder is governed by the larger value for one design lane versus two or more design lanes loaded. Further, the live load distribution factors for the positive and negative moments are different. Therefore, a total of four cases are evaluated which are presented as follows:

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- One design lane loaded – positive moment:

Distribution factor for moment (*DFM*) can be calculated by:

$$DFM = k\left(\frac{b}{33.3L}\right)^{0.5}\left(\frac{I}{J}\right)^{0.25} \qquad \text{(AASHTO Table 4.6.2.2.2b-1)}$$

Provided that the ranges are met:

$35 \le b \le 60$

b = girder width = 48 inches (OK)

$20 \le L \le 120$

L = span of girder = 97 feet (OK)

$5 \le N_b \le 20$

N_b = number of girders = 15 (OK)

where:

$$k = 2.5(N_b)^{-0.2} \ge 1.5$$

$$\because k = 2.5(15)^{-0.2} = 1.45 < 1.5 \ (\text{NG}), \text{ use } k = 1.5$$

$$\frac{I}{J} = 0.54\left(\frac{d}{b}\right) + 0.16 \qquad \text{(AASHTO Table 4.6.2.2.1-3)}$$

$$\frac{I}{J} = 0.54\left(\frac{39}{48}\right) + 0.16 = 0.60$$

$$DFM = 1.5\left(\frac{48}{33.3(97)}\right)^{0.5} = (0.60)^{0.25} = 0.161 \text{ lanes/girder}$$

- One design lane loaded – negative moment:

In this case, the average lengths of Spans 1 and 2 are used.

$$L = \frac{75+100}{2} = 87.5 \text{ ft}$$

$$DFM = 1.5\left(\frac{48}{33.3(87.5)}\right)^{0.5}(0.60)^{0.25} = 0.169 \text{ lanes/girder}$$

- Two or more design lanes loaded – positive moment:

$$DFM = k\left(\frac{b}{305}\right)^{0.6}\left(\frac{b}{12.0L}\right)^{0.2}\left(\frac{I}{J}\right)^{0.06} \qquad \text{(AASHTO Table 4.6.2.2.2b-1)}$$

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$$DFM = 1.5\left(\frac{48}{305}\right)^{0.6}\left(\frac{48}{12.0(97)}\right)^{0.2}(0.60)^{0.06} = 0.253 \text{ lanes/girder}$$

- Two or more design lanes loaded – negative moment:

$$DFM = 1.5\left(\frac{48}{305}\right)^{0.6}\left(\frac{48}{12(87.5)}\right)^{0.2}(0.60)^{0.06} = 0.259 \text{ lanes/girder}$$

In summary, 0.253 lanes per girder will be used for the flexure design of interior girders of Span 2 under positive bending and 0.259 lanes per girder under negative bending.

Live Load Shear Distribution Factor (Interior Girders)

The live load distribution factor for shears (*DFV*, lanes/girder), for an interior girder is governed by the larger value for one lane versus two lanes loaded, as shown below. Note that according to AASHTO Table 4.6.2.2.1-2, the value of *L* used for determining the distribution factor for shears (*DFV*) is the same as that for *DFM*

- One lane loaded:

  DFV is calculated by:

$$DFV = \left(\frac{b}{130L}\right)^{0.15}\left(\frac{I}{J}\right)^{0.05} k_q k_\theta \qquad \text{(AASHTO Table 4.6.2.2.3a-1)}$$

  where the skew magnification factor (k_θ) is applied to all girders per CA Amendments Table 4.6.2.2.3c-1.

$$k_\theta = 1 + 12\ L\ (\tan\theta)^{0.5} / (90d)$$

$$= 1 + 12\ (97)(\tan\ 10)^{0.5} / (90\ x\ 39) = 1.14$$

  Provided that the ranges for flexure and additional ranges below are met:

  25,000 ≤ *J* ≤ 610,000

  From the above calculation, the ratio of *I/J* is 0.60. Therefore, *J* is calculated as:

$$J = \frac{I}{0.60} = \frac{111{,}829}{0.60} = 186{,}382 \text{ in.}^4$$

  which is within the acceptable range.

  40,000 ≤ *I* ≤ 610,000

$$I = 111{,}829 \text{ in.}^4 \qquad \text{(OK)}$$

$$DFV = 0.489 \text{ lanes/girder}$$

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- Two or more design lanes loaded:

$$DFV = \left(\frac{b}{156}\right)^{0.4} \left(\frac{b}{12.0L}\right)^{0.1} \left(\frac{I}{J}\right)^{0.05} \left(\frac{b}{48}\right) k_{\theta}^{0.4} \quad \text{(AASHTO Table 4.6.2.2.3a-1)}$$

with $\left(\frac{b}{48}\right) = 1.0$

$$DFV = \left(\frac{48}{156}\right)^{0.4} \left(\frac{48}{12.0(97)}\right)^{0.1} (0.60)^{0.05} (1.00)\ (1.14) = 0.504 \text{ lanes/girder}$$

Therefore, *DFV* for two or more lanes loaded is larger, and this controls.

Use *DFV* = 0.504 lanes/girder

Note: The dynamic load allowance factor (*IM*) is applied to the HL93 design truck, design tandem, and P15 permit truck only, not to the HL93 design lane load. Table 3.6.2.1-1 of California Amendments (Caltrans, 2019) 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. In this example, structure analysis software was used to determine the live load moments and shears. The results are tabulated in Tables 5.4-5 and 5.4-6 for HL93 loading and Tables 5.4-7 and 5.4-8 for P15 loading, respectively. These tables list the envelope values for unfactored live load moment and shear per lane, as well as per girder for a design using distribution factors. The P15 values are derived from single 54-kip axle loads versus 27-kip tandem bogies, as shown in California Amendment Figure 3.6.1.8.1.

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Table 5.4-5 Unfactored Live Load Moments – HL93

Location		Per Lane†		Per Girder	
Dist/Span (*X/L*)	Location (ft)	Positive Moment (kip-ft)	Negative Moment (kip-ft)	Positive Moment (kip-ft)	Negative Moment (kip-ft)
Bearing	0	226	-1,636	58.0	-428
H/2*	1.63	228	-1,490	58.5	-390
Transfer**	2.5	232	-1,416	59.5	-371
0.1*L*	9.7	360	-943	92.3	-247
0.2*L*	19.4	823	-598	211	-157
0.3*L*	29.1	1,312	-517	336	-135
0.4*L*	38.8	1,614	-441	414	-115
0.5*L*	48.5	1,702	-365	437	-96

*H/2 = Assumed location for critical shear
** 60 strand diameters from the end of the girder
†From a structural analysis program (Includes *IM* = 33%)

Table 5.4-6 Unfactored Live Load Shears Per Lane and Corresponding Moments – HL93

Location		Per Lane†		Per Girder	
Dist/Span (*X/L*)	Location (ft)	Shear (kip)	Moment (kip-ft)	Shear (kip)	Moment (kip-ft)
Bearing	0	119	-668	61.3	-175
H/2*	1.63	117	-561	60.3	-147
Transfer**	2.5	116	-504	59.8	-132
0.1*L*	9.7	107	-60.1	55.1	-16
0.2*L*	19.4	91.0	574	46.9	147
0.3*L*	29.1	75.2	1,014	38.8	260
0.4*L*	38.8	59.8	1,244	30.8	319
Midspan	48.5	47.3	1,352	24.4	347

**H/2* = Assumed location for critical shear
**60 strand diameters from the end of the girder
†From a structural analysis program (Includes *IM* = 33%)

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Table 5.4-7 Unfactored Live Load Moments – P15

Location		Per Lane†		Per Girder	
Dist/Span (*X/L*)	Location (ft)	Positive Moment (kip-ft)	Negative Moment (kip-ft)	Positive Moment (kip-ft)	Negative Moment (kip-ft)
Bearing	0	354	-2,856	90.8	-748
H/2*	1.63	358	-2,590	91.8	-678
Transfer**	2.5	365	-2,455	93.6	-643
0.1*L*	9.7	526	-1,592	135	-417
0.2*L*	19.4	1,097	-935	281	-245
0.3*L*	29.1	1,981	-776	508	-203
0.4*L*	38.8	2,521	-627	647	-164
Midspan	48.5	2,709	-520	695	-136

*H/2 = Assumed location for critical shear
**60 strand diameters from the end of the girder
†From a structural analysis program (Includes *IM* = 25%)

Table 5.4-8 Unfactored Live Load Shear and Corresponding Moments – P15

Location		Per Lane†		Per Girder	
Dist/Span (*X/L*)	Location (ft)	Shear (kip)	Moment (kip-ft)	Shear (kip)	Moment (kip-ft)
Bearing	0	224	-2,491	115	-652
H/2*	1.63	217	-2,165	112	-567
Transfer**	2.5	214	-1,997	110	-523
0.1*L*	9.7	186	-885	95.9	-232
0.2*L*	19.4	156	127	80.4	33
0.3*L*	29.1	128	972	66.0	249
0.4*L*	38.8	99	1,382	51.0	354
Midspan	48.5	77	1,466	39.7	376

*H/2 = Assumed location for critical shear
**60 strand diameters from the end of the girder
†From a structural analysis program (Includes *IM* = 25%)

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5.4.7.8 Determine Prestressing Force and Area of Strands

The preliminary jacking force, P_j, and the associated area of prestressing strands can be calculated by satisfying the two tensile stress limits at the bottom fiber of the girder at the Service III limit state, which include the following cases:

Case A: No tension is allowed for components with bonded prestressing tendons or reinforcement, subject to permanent loads (*DC*, *DW*) only. Set the stress at the bottom fiber equal to zero and solve for the required, *P*, to achieve no tension. Note that the compressive stress is positive (+) and the tensile stress is negative (-) in this example.

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

Rearranging the equation:

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

As shown in Table 5.4-4 (*DC* and *DW*) and Table 5.4-5 and 5.4-6 (HL93 vehicular live load), the maximum moment due to dead loads and live loads occurs at the midspan.

The following moments at the midspan are used from Table 5.4-4 for the girder design where:

Moment due to the self-weight of the girder and the CIP concrete deck

M_{DC1} = 1,297 kip-ft

Moment due to the weight of the barriers and the sidewalk

M_{DC2} = 43.6 kip-ft

Moment due to the future wearing

M_{DW} = 52.6 kip-ft

Girder section modulus (bottom fiber) from Section 5.4.7.5

S_b = 6,778 in.^{3}

Girder cross section area from Section 5.4.7.5

A_g =766 in.^{2}

In order to evaluate loading effects on the entire bridge, the composite section properties are calculated by the parallel axis theorem, in which the cross area of the CIP deck with a lower modulus of elasticity is transferred to the higher strength girder concrete by the modular ratio (*n*).

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Table 5.4-9 Gross Section Properties (Composite)

Section	Transformed Area, A_i (in.^{2})	y_i (in.)	$A(y_i)$ (in.^{3})	$A(y_i)^2$ (in.^{4})	I_o (in.^{4})
Girder	766	16.50	12,639	208,544	111,829
Deck	272	36	9,782	352,169	815
Total	1038		22,421	560,712	112,645

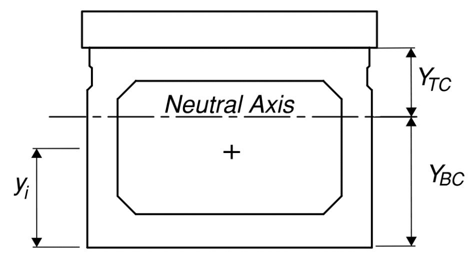
A_C = 1,038 in.^{2}

$Y_{BC} = \sum A_i y_i \div A_C$ = 22,421 ÷ 1,038 = 21.61 in.

$Y_{TC} = D - Y_{BC}$ = 33.00 - 21.61 = 11.39 in.

$I_C = \sum I_o + \sum A_i y_i^2 - A_C Y_{BC}^2$ = 188,915 in.^{4}

$S_{BC} = I_C \div Y_{BC}$ = 188,915 ÷ 21.61 = 8,744 in.^{3}



where:

$$n = \frac{E_{deck}}{E_{girder}} = \frac{3,987}{4,291} = 0.929$$

Transformed deck width = n (girder spacing) = 0.929(48.75) = 45.29 in.

Transformed deck area = 45.29 (6) = 271.74 in.^{2}

Transformed deck moment of inertia $= 45.3\frac{6^3}{12} =$ 815 in.^{4}

y_i = distance from the centroid of the section i to the girder bottom fiber

A_c = concrete area of the composite section

Y_{BC} = distance from the composite section neutral axis to the girder bottom fiber

Y_{TC} = distance from the composite neutral axis to the girder top fiber

I_C = moment of inertia of the composite section

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

To determine the effective prestressing force (P), an estimate of the eccentricity of the strand relative to the non-composite girder (e_c) is required. This estimate is calculated using the concrete cover to the stirrups, sizes of stirrups, and diameters of prestressing strands. For this example, non-corrosive exterior exposure is assumed, which corresponds to 1.5 inches of the cover from the soffit, per Table 5.10.1-1 of California

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Amendments (Caltrans, 2019). The stirrups are comprised of #5 bars and the prestressing strand is 0.6 inches in diameter. Use deformation diameter of reinforcing steel when determining clearances.

$$\text{Strand distance from the bottom of the girder} = 1.5 + 0.69 + \frac{0.6}{2} = 2.49 \text{ in.}$$

Round up to the nearest 0.5-inches, use 2.5 inches for the lowest row of strands. Assume 24 strands in two rows, with 22 strands in the lowest row and two strands in the upper. (See Figure 5.4-12). The distance from the centroid of the strands to the bottom of the girder is referred to as the offset. Also assuming 2-inch spacing between rows, the

$$\text{Strand offset} = \frac{2(4.5) + 22(2.5)}{24} = 2.67 \text{ in.}$$

$$e_c = y_b - \text{ offset } = 16.5 - 2.67 = 13.8 \text{ in.}$$

$$P = \frac{\dfrac{1{,}297\,x12}{6{,}778} + \left(\dfrac{(43.6 + 52.6)(12)}{8{,}744}\right)}{\dfrac{1}{766} + \dfrac{13.8}{6{,}778}}$$

Required prestressing force, P = 726 kips

Case B: Tension stress limit for components subject to the Service III limit state (*DC*+ *DW*+ (0.8) HL93), subjected to not worse than moderate corrosion conditions, and are located in Caltrans Environmental “non-freeze-thaw area” from Table 5.9.2.3.2b-1 of California Amendments (Caltrans, 2019). is:

$$\text{Stress Limit} = -0.19\lambda\sqrt{f'_c} = -\ 0.19\ x\ 1\ x\sqrt{5} = -0.424 \text{ ksi}$$

Note: λ= 1.0 for normal weight concrete

$$\frac{P}{A_g} + \frac{Pe_c}{S_b} - \frac{M_{DC1}}{S_b} - \frac{M_{DC2} + M_{DW} + 1.0(M_{HL93})}{S_{BC}} == -0.19\sqrt{f_c'}$$

where:

M_{HL93} = moment due to HL93 loading at midspan = 437 kip-ft (Table 5.4-5)

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Rearranging the equation:

$$P = \frac{\frac{M_{DC1}}{S_b} + \left(\frac{M_{DC2} + M_{DW} + 1.0(M_{HL93})}{S_{BC}}\right) - (0.19)\sqrt{f'_c}}{\frac{1}{A_g} + \frac{e_c}{S_b}}$$

$$= \frac{\frac{1{,}297\text{x}12}{6{,}778} + \left(\frac{(43.6+52.6+1.0(437))(12)}{8{,}744}\right) - (0.19)\sqrt{5}}{\frac{1}{766} + \frac{13.8}{6{,}778}} = 776 \text{ kips}$$

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

$P_f = P = 776$ kips/girder.

To determine the minimum required jacking force, an estimate of the 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 the transfer, ignoring minor losses from jacking to de-tensioning) is:

The minimum Jacking Force, $P_j = \frac{776}{0.75} = 1{,}034$ kips

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

$$A_{ps} = \frac{1{,}034}{0.75(270)} = 5.11 \text{ in.}^2$$

Number of 0.6 in. diameter strands $= \frac{5.11}{0.217} = 23.5$

Rounding to an even number, 24 strands will provide symmetry (about a vertical line through the centroid) to produce uniform stress distribution in the member.

Therefore, using 24 0.6-inch diameter low-relaxation Grade 270 strands, the prestressing strand cross section area is:

$A_{ps} = 24(0.217) = 5.21$ in.^{2}

The total prestressing force at jacking, $P_j = 0.75(270)(5.21) = 1{,}055$ kips

5.4.7.9 Estimate Prestress Losses

The total prestress loss is the sum of immediate and long-term losses. Immediate losses for strands in a precast box girder are due to the elastic shortening. Long-term losses are primarily due to concrete creep and shrinkage as well as steel relaxation.

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

where:

Δf_{pES} = elastic shortening loss (ksi)

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

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

5.4.7.9.1 Elastic Shortening

The loss due to the elastic shortening, $\Delta f_{pES,}$ in precast concrete box girders can be determined by AASHTO-CA BDS-8 Eq. (C5.9.3.2.3a-1) as follows:

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

where:

A_{ps} = area of prestressing steel = 5.21 in.^{2}

A_g = gross area of girder = 766 in.^{2}

f_{pbt} = stress in prestressing steel immediately prior to transfer and ignoring minor relaxation losses after jacking (ksi)

= 0.75(270) = 202.5 ksi,

E_{ci} = 3,987 ksi

E_p = 28,500 ksi

e_m = prestressing steel eccentricity at the midspan = 13.8 in.

I_g = moment of inertia of gross section =111,829 in.^{4}

M_g = girder midspan moment, self-weight only

= $0.125\, w_g\, L^2 = 0.125(0.798)97^2(12) = 11{,}261$ k-in.

$$\Delta f_{pES} = \frac{5.21(202.5)\left(111{,}829+13.8^2(766)\right)-13.8(11{,}261)(766)}{5.21\left(111{,}829+13.8^2(766)\right)+\dfrac{766(111{,}829)(3{,}987)}{28{,}500}} = 12.8 \text{ ksi}$$

The initial prestress force immediately after the transfer

P_i = 202.5 – 12.8 = 189.7 ksi

As stated in Article C5.9.3.2.3a when calculating concrete stresses using transformed section properties, the effects of losses due to the elastic deformation are implicitly accounted for, and Δf_{pES} should not be included in the prestressing force applied to the transformed section at the transfer.

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5.4.7.9.2 Long Term Losses

There are two methods to estimate the time-dependent prestress losses: the approximate method, per Article 5.9.3.3, and the refined method, per Article 5.9.3.4. The approximate method is appropriate for the precast box girder provided the conditions of Article 5.9.5.3 are met. These conditions require the normal load and environmental conditions, where:

- Normal-weight concrete is used (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)

Since all conditions of the precast box girder design example are satisfied, the approximate method is 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:

$$\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR} \qquad \text{(AASHTO 5.9.3.3-1)}$$

where:

Δf_{pLT} = prestress losses

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

= 202.5 ksi

H = average annual ambient relative humidity (%) = 70%

γ_h = correction factor for a relative humidity of ambient air

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

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

= $5/(1+ f'_{ci}) = 5/(1+4.0) = 1.0$ (AASHTO 5.9.3.3-3)

Δf_{pR} = an estimation of relaxation loss taken as 2.4 ksi for low relaxation strand and in accordance with manufacturers recommendation for other types of the strand (ksi).

Therefore,

$$\Delta f_{pLT} = 10.0\frac{(202.5)(5.21)}{766}(1)(1) + 12.0(1)(1) + 2.4 = 28.2 \text{ ksi}$$

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$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} = 12.8 + 28.2 = 41.0 \text{ ksi}$$

$$\Delta f_{pT} = \frac{41.0}{202.5}(100\%) = 20.2\%$$

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

$$\text{Elastic gains} = n\left(\frac{M_{DC1}' \times e_m}{I_g} + \frac{\left(M_{DC2} + M_{DW} + M_{LL}\right)e_c}{I_c}\right)$$

where:

M_{DC1}' = Moment at midspan due to the deck weight

= $\dfrac{w_S \times L^2}{8} = \dfrac{0.305 \, x \, 97^2}{8} = 358$ kip-ft

M_{DC2} = 43.6 kip-ft (Table 5.4-4)

M_{DW} = 52.6 kip-ft (Table 5.4-4)

M_{LL} = 437 kip-ft (Table 5.4-5)

n = $\dfrac{E_{ps}}{E_c} = \dfrac{28{,}500}{4{,}291} = 6.64$

e_m = strand eccentricity of the girder = 13.8 in.

e_c = strand eccentricity of the composite section

= Y_{BC} – offset = 21.6 – 2.7 = 18.9 in.

Elastic gains

$$\text{Elastic gains} = 6.64\left(\frac{358 \times 12 \times 13.8}{111{,}829} + \frac{12\left(43.6 + 52.6 + 437\right)18.9}{188{,}915}\right) = 7.8 \text{ ksi}$$

f_{pe} = effective stress in prestressing strands using gross non-transformed section properties (service limit state)

= 202.5 – 41.0 + 7.8 = 169.3 ksi

Check prestressing stress limit at service limit state:

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

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0.8 (270) (0.9) = 194.4 ksi > 169.3 ksi, (OK)

P_f = the effective prestressing force after all losses using gross, non-transformed sections (kip)

= $A_{ps} \times f_{pe}$ = 5.21 (169.3) = 882 kips

Transformed sections will be used in subsequent sections of this design example, and the prestressing force at the transfer is the jacking force, Pj, without the elastic shortening loss. The prestressing force used in concrete stress calculations at the service includes the long-term loss only, per AASHTO Article C5.9.3.3.

f_{pe} = effective stress in the prestressing strand at the service

= $0.75\ f_{pu} - \Delta f_{pLT}$

= 0.75 (270) – 28.2 = 174.3 ksi

P_f = effective prestress force at the service

= $f_{pe}\ (A_{ps})$

= 174.3(5.21) = 908 kips

5.4.7.10 Design for Service Limit State

Design for the Service Limit State addresses the suitability of the previously estimated prestressing force and the profile based on the loading stages presented in Section 5.4.7.3. Concrete stresses are checked at the transfer, which may lead to design modifications such as adjusting the strand profile or the initial concrete compressive strength, f'_{ci}. Normally the check of the tensile stress at the bottom of the girder is critical to prevent possible cracking at Service III (HL93 vehicular live load).

5.4.7.10.1 Calculate Transformed Section Properties

The use of transformed section properties generally leads to more accurate calculations than the use of gross section properties because it accounts for the stiffness of the prestressing strand, as discussed in AASHTO Article C5.9.3.2.3a.

Three sets of transformed section properties are needed for the service limit state design. These include: (1) transformed section properties of the girder at the transfer (Stage I), (2) transformed section properties of the girder immediately prior to the deck casting (Stage IIA), and (3) transformed composite section properties at the service (Stage III). These section properties at each stage are calculated separately because of the difference in the final and the initial transformed non-composite properties, e.g., E_c versus E_{ci}.

The prestressing strand steel area A_{ps} is transformed to the concrete by multiplying (n-1), where n is the modular ratio of the prestressing strand to concrete at the transfer that is given by:

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$$n = \frac{E_{ps}}{E_{ci}} = \frac{28{,}500}{3{,}987} = 7.15$$

The transformed strand area, then, is calculated as:$(n\text{-}1)\ A_{ps}$ = (7.15 – 1) 5.21 = 32.0 in.^{2}

Table 5.4-10 Transformed Section Properties at Transfer

Section	Transformed Area, A_i (in.^{2})	y_i (in.)	$A(y_i)$ (in.^{3})	$A(y_i)^2$ (in.^{4})	I_o (in.^{4})
Girder	766	16.5	12,639	208,544	111,829
Strand	32	2.67	85	228	0
Total	798		12,724	208,771	111,829

A_C = 798 in.^{2}

$Y_{Bti} = \sum Ay_i \div A_C$
= 12,724 ÷ 798 = 15.9 in.

$Y_{Tti} = D - Y_{Bti}$ = 33.0 - 15.9
= 17.1 in.

$I_{ti} = \sum I_o + \sum Ay_i^2 - A_C\ Y_{Bti}^2$
= 117,712 in.^{4}

$S_{Bti} = I_{ti} \div Y_{Bti}$
= 117,712 ÷ 15.9 = 7,382 in.^{3}

$S_{Tti} = I_{ti} \div Y_{Tti}$
= 117,712 ÷ 17.1= 6,902 in.^{3}

$e_{ti} = Y_B - y_{i(strand)}$
= 15.94 - 2.67 = 13.3 in.

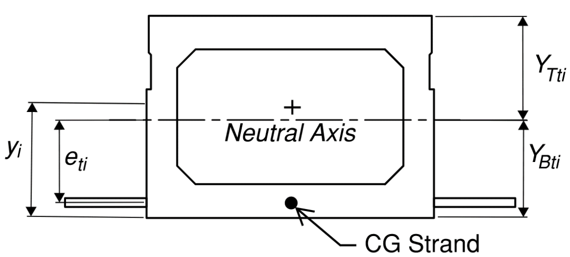

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Table 5.4-11 Transformed Section Properties at Final (Non-composite)

Section	Transformed Area, A_i (in.^{2})	y_i (in.)	$A(y_i)$ (in.^{3})	$A(y_i)^2$ (in.^{4})	I_o (in.^{4})
Girder	766	16.5	12,638	208,544	111,829
Strand	29.4	2.67	78	209	0
Total	795		12,717	208,752	111,829

Y_{Btf} = 12,717/795 = 16.0 in.

Y_{Ttf} = 33.0 – 16.0 = 17.0 in.

$I_{tf} = \sum I_o + \sum A y_i^2 - A_C\, Y_{Btf}^2$ = 117,244 in.^{4}

S_{Btf} = 117,244/16.0 = 7,333 in.^{3}

S_{Ttf} = 117,244/17.0 = 6,892 in.^{3}

e_{tf} = 16.0 - 2.67 = 13.3 in.

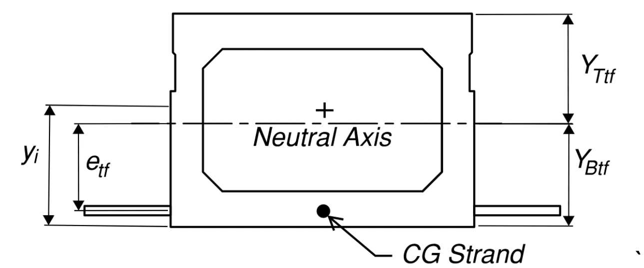


Table 5.4-12 Transformed Section Properties at Final (Composite)

Section	Transformed Area, A_i (in.^{2})	y_i (in.)	$A(y_i)$ (in.^{3})	$A(y_i)^2$ (in.^{4})	I_o (in.^{4})
Girder	795	16.0	12,717	203,338	117,244
Deck	272	36	9,782	352,169	815
Total	1,067		22,500	555,506	118,059

Y_{BCt} = 22,500 ÷ 1067 = 21.1 in.

Y_{TGCt} = 33.0 – 21.1 = 11.9 in.

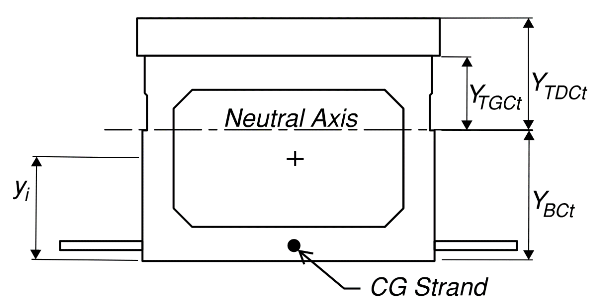
Y_{TDCt} = 39.0 – 21.1 = 17.9 in.

$I_{Ct} = \sum I_o + \sum A y_i^2 - A_C\, Y_{BCt}^2$ = 199,164 in.^{4}

S_{BCt} = 199,164 ÷ 21.1 = 9,446 in.^{3}

S_{TGCt} = 199,164 ÷ 11.9 = 16,715 in.^{3}

S_{TDCt} = 199,164 ÷ 17.9 = 11,117 in.^{3}



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The modulus ratio at the final service condition is:

$$n = \frac{E_{ps}}{E_c} = \frac{28,500}{4,291} = 6.64$$

Transformed strand area, then, is calculated as:

$(n - 1)\,A_{ps} = (6.64-1)5.21 = 29.4 \text{ in.}^2$

5.4.7.10.2 Concrete Stress Check at Transfer Condition

The check of concrete stresses at the transfer investigates the suitability of both the prestressing force and the strand profile for the assumed section. Strands are straight in precast box girders. Concrete stresses shall be checked to ensure stress limits for concrete are not exceeded. Since all prestress is applied to the beam initially with only a fraction of the weight applied and the strands are straight, stresses need only be checked at the ends of the girders.

Compressive stress limit = $0.65f'_{ci}$ = 0.65 (4.0) = 2.60 ksi (AASHTO 5.9.2.3.1a)

Tensile stress limit without bonded auxiliary reinforcement:

Stress Limit $= 0.0948\lambda\sqrt{f'_{ci}} \leq 0.200$ ksi (AASHTO Table 5.9.2.3.1b-1)

$$= 0.0948(1.0)\sqrt{4.0} = 0.190 \text{ ksi}$$

Tensile stress limit in areas with bonded auxiliary reinforcement sufficient to resist the tensile force:

Stress Limit $= 0.24\lambda\sqrt{f'_c}$ (AASHTO Table 5.9.2.3.1b-1)

$$= 0.24(1.0)\sqrt{4.0} = 0.480 \text{ ksi}$$

The elastic shortening loss, Δf_{pES}, is not to be subtracted from the strand stress in calculating the prestressing force at the transfer (taken as P_j because relaxation losses between the jacking and the transfer are ignored), as discussed in Section 5.4.7.9.

Check concrete stresses at the transfer length (X) from the ends of the girders to account for the development of the strand into the girder.

X = $60(d_b)$ = 60 (0.6) = 36.0 in. (AASHTO 5.9.4.3.1)

d_b = nominal strand diameter (in.)

P_j = 1,055 kips

A_{ti} = transformed cross section area of the girder at transfer = 798 in.^{2}

e_{ti} = strand eccentricity = 13.3 in.

S_{Bti} = section modulus with bottom fiber in tension at transfer

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= 7,382 in.^{3}

S_{Tti} = section modulus with top fiber in tension at transfer = 6,902 in.^{3}

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

$$= A_g w_c \left(\frac{LX}{2} - \frac{X^2}{2} \right)$$

where:

A_g = gross girder cross section area = 766 in.^{2}

w_c = unit weight of concrete including the weight of reinforcement, prestress, etc., for loads only = 0.150 k/ft^3

L = effective span length at transfer = 98 ft

(For effective length to calculate girder selfweight, is assumed that the girder is supported at 1'-0" from ends)

$$M_{DC1} = \frac{766 \text{ x } 0.15}{144} \left(\frac{98 \text{ x } 3}{2} - \frac{3^2}{2} \right) = 114 \text{ kip-ft} = 1{,}364 \text{ kip-in.}$$

Calculate concrete stress at the top of the girder at transfer length.

$$f_{top} = \frac{P_j}{A_{ti}} - \frac{P_j e_{ti}}{S_{Tti}} + \frac{M_{DC1}}{S_{Tti}}$$

$$f_{top} = \frac{1{,}055}{798} - \frac{1{,}055(13.3)}{6{,}902} + \frac{1{,}364}{6{,}902} = -0.510 \text{ ksi (Tension)}$$

The tensile stress exceeds the upper limit of the tensile stress limit for areas with the bonded reinforcement (-0.480 ksi).

$$f_{bot} = \frac{P_j}{A_{ti}} + \frac{P_j e_{ti}}{S_{Bti}} - \frac{M_{DC1}}{S_{Bti}}$$

$$f_{bot} = \frac{1{,}055}{798} + \frac{1{,}055(13.3)}{7{,}382} - \frac{1{,}364}{7{,}382} = 3.03 \text{ ksi (Compression)}$$

The compressive stress limit of 2.60 ksi is exceeded. The designer can either increase the allowable compressive stress or debond the strands. Since the stress limit is exceeded at one location, debonding strands is the most economical choice.

The maximum number of strands that can be debonded is 33%, per AASHTO Article 5.11.4.3 and California Amendment 5.9.4.3.3 (Caltrans, 2019). An even number of strands should be debonded to maintain symmetry. Debonding six strands is evaluated.

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$$\frac{6}{24}\times 100 = 25\% < 33\% \left(\text{OK}\right)$$

The force in the prestressing strand in the debonded zone is calculated as:

$$P_j = \left(24 - 6\right)\left(0.217\right)\left(202.5\right) = 791 \text{ kip}$$

$$f_{top} = \frac{791}{798} - \frac{791\left(13.3\right)}{6,902} + \frac{1,343}{6,902} = -0.333 \text{ ksi, within limit of -0.480 ksi} \left(\text{OK}\right)$$

$$f_{bot} = \frac{791}{798} + \frac{791\left(13.3\right)}{7,382} - \frac{1,343}{7,382} = 2.23 \text{ ksi} < 2.60 \text{ ksi} \left(\text{OK}\right)$$

Debonding six strands at the ends reduces temporary stresses within stress limits. At the top fiber, the tensile stress limit for regions without bonded reinforcement is exceeded. Therefore, reinforcement in the top fiber is provided and is checked in the subsequent text.

The top fiber stress exceeds 0.19 ksi at the transfer length from the end of the girder. Therefore, bonded auxiliary reinforcement at the top of the girder is required. The amount of bonded reinforcement is based on the total tension force in the concrete at the top girder. This force is the area of the concrete in tension multiplied by the average tension stress, since the stress is linearly distributed over the depth, as shown in Figure 5.4-13.

The depth of the area in tension (x) is calculated as:

$$x = h_{girder}\frac{\left|f_{top}\right|}{\left|f_{top}\right| + \left|f_{bot}\right|} = \left(33\right)\frac{0.333}{0.333 + 2.23} = 4.29 \text{ in.}$$

which is less than the thickness of the flange (5.5 in.).

The tensile force (T) is calculated as:

$$T = \frac{f_{top}}{2}bx = \frac{0.333}{2}\left(48\right)\left(4.29\right) = 34.3 \text{ kips}$$ (AASHTO C5.9.2.3.1b)

Assuming stress in the reinforcement of 30 ksi (also, f_s=0.5f_y per AASHTO C5.9.2.3.1b), the area of bonded reinforcement (A_s) is calculated as:

$$A_s = \frac{T}{f_s} = \frac{34.3}{30} = 1.14 \text{ in.}^2$$

A total 5 - #5, A_s = 1.55 in.^{2} is specified for distribution reinforcement, which exceeds the tension requirement by a significant amount. Bars will run full length on the top of the girder.

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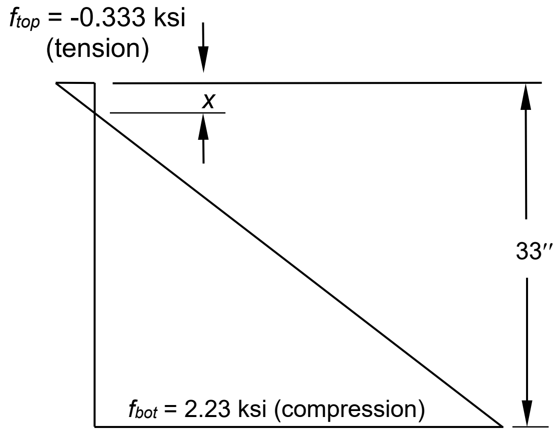

Figure 5.4-13 Concrete Stress Distribution at the Transfer Section

The length of debonding is initially assumed, and the top and bottom girder stresses at the end of the debonding influence are calculated and compared to the stress limits. Debonding influence extends to the end of the debonding plus the transfer length of 60 d_b. At this location, the jacking force of all strands is effective and the moments include the self-weight.

Assuming a debonding length of 8.0 feet,

$$X = 8.0 + 60d_b = 8.0 + 3.0 = 11.0 \text{ ft}$$

$$M_{DC1} = \frac{766 \times 0.15}{144}\left(\frac{98 \times 11}{2} - \frac{11^2}{2}\right) = 382 \text{ kip-ft} = 4{,}581 \text{ kip-in.}$$

$$f_{top} = \frac{1{,}055}{798} - \frac{1{,}055(13.3)}{6{,}902} + \frac{4{,}581}{6{,}902} = -0.044 \text{ ksi, within limit of -0.480 ksi (OK)}$$

$$f_{bot} = \frac{1{,}055}{798} + \frac{1{,}055(13.3)}{7{,}382} - \frac{4{,}581}{7{,}382} = 2.60 \text{ ksi = 2.60 ksi (OK)}$$

This and the prior calculations demonstrate that stresses within and beyond the influence of the debonding are within stress limits without auxiliary reinforcement.

5.4.7.10.3 Concrete Stress Check at Service Limit State

The check of concrete stresses at the service limit state investigates the suitability of the section to resist service loads. The goal is to prevent flexural cracking of the section at the midspan for Service Limit III due to the HL93 vehicular Live Load and ensure that the girder stresses do not exceed a stress limit which could cause microcracking to develop. In addition, the California Amendments (Caltrans, 2019) prohibit tension under permanent loads.

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The compressive stress limit due to unfactored permanent loads (including girder, CIP concrete deck, sidewalks barrier, and future wearing surface) and prestressing force is:

$$= 0.45f'_c = 0.45(5.0) = 2.25 \text{ ksi} \qquad \text{(AASHTO Table 5.9.2.3.2a-1)}$$

The compressive stress limit due to effective prestress, permanent, and transient loads, which includes all dead and live loads is:

$$= 0.60\phi_w f'_c = 0.60(1.0)(5.0) = 3.00 \text{ ksi} \qquad \text{(AASHTO Table 5.9.2.3.2a-1)}$$

The tensile stress limit for pre-compressed tensile regions with bonded prestressing tendons subjected to permanent loads only, which includes prestress and dead loads, is:

$$\text{Tensile Limit} = 0 \text{ ksi (no tension allowed)} \qquad \text{(CA Table 5.9.2.3.2b-1)}$$

The tensile stress limit for pre-compressed tensile regions with bonded prestressing tendons subjected to Service III loading conditions, i.e., *PS + Perm + 0.8(LL+IM)*$_{HL-93}$, which includes all permanent and transient loading, is:

$$\text{Tensile Limit} = 0.19\lambda\sqrt{f'_c} = 0.19(1.0)\sqrt{5.0} = 0.424 \text{ ksi} \qquad \text{(CA Table 5.9.2.3.2b-1)}$$

Concrete stresses are checked at the midspan for Cases A and B, previously described in Section 5.4.7.8.

Case A: unfactored permanent and prestress loads only.

$$f_{top} = \frac{P}{A_g} - \frac{Pe_c}{S_t} + \frac{M_{DC1}}{S_t} + \left(\frac{M_{DC2} + M_{DW}}{S_{tc}}\right)$$

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

where:

P	=	908 kip	(Section 5.4.7.9.2)
M_{DC1}	=	1,297 kip-ft	(Table 5.4-4)
M_{DC2}	=	43.6 kip-ft	(Table 5.4-4)
M_{DW}	=	52.6 kip-ft	(Table 5.4-4)
M_{LL+I}	=	437 kip-ft	(Table 5.4-5)
S_b	=	transformed girder section modulus – bottom fiber (non-composite) = 7,333 in.^{3}	(Table 5.4-11)
S_t	=	transformed girder section modulus – top fiber (non-composite) 6,892 in.^{3}	(Table 5.4-11)

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S_{BC} = transformed girder section modulus – bottom fiber (composite) = 9,446 in.^{3} (Table 5.4-12)

S_{tc} = transformed girder section modulus – top fiber (composite) = 16,715 in.^{3} (Table 5.4-12)

A_g = transformed girder area (non-composite) = 795 in.^{2} (Table 5.4-12)

e_c = strand eccentricity (non-composite) = 13.3 in. (Table 5.4-11)

$$f_{top} = \frac{908}{795} - \frac{908 \times 13.3}{6{,}892} + \frac{12 \times 1{,}297}{6{,}892} + \left(\frac{12(43.6 + 52.6)}{16{,}715}\right)$$

$$= 1.71 \text{ ksi} < 2.25 \text{ ksi} (\text{OK})$$

$$f_{bottom} = \frac{908}{795} + \frac{908 \times 13.3}{7{,}333} - \frac{12 \times 1{,}297}{7{,}333} - \left(\frac{12(43.6 + 52.6)}{9{,}446}\right)$$

$$= 0.547 \text{ ksi} > 0 \text{ ksi} (\text{OK})$$

Case B-I: Concrete stress at Service I Limit state. Note that only compressive stresses in the top fiber of the girder are evaluated.

$$f_{top} = \frac{P}{A_g} - \frac{Pe_c}{S_t} + \frac{M_{DC1}}{S_t} + \left(\frac{M_{DC2} + M_{DW} + M_{LL+I}}{S_{tc}}\right)$$

$$= \frac{908}{795} - \frac{908 \times 13.3}{6{,}892} + \frac{12 \times 1{,}297}{6{,}892} + \left(\frac{12(43.6 + 52.6 + 437)}{16{,}715}\right)$$

$$= 2.02 \text{ ksi} < 3.00 \text{ ksi} (\text{OK})$$

Case B-III: Concrete stress at Service III Limit State with unfactored permanent, prestress, and 100% of the HL93 vehicular load, as is required in Table 3.4.1.-4 of AASHTO, where the elastic gain is accounted for in the use of transformed sections. Note that only the tensile stresses in the bottom fiber of the girder are evaluated.

$$f_{bottom} = \frac{P}{A_g} + \frac{Pe_c}{S_b} - \frac{M_{DC1}}{S_b} - \left(\frac{M_{DC2} + M_{DW} + 1.0M_{LL+I}}{S_{BC}}\right)$$

$$= \frac{908}{795} + \frac{908 \times 13.3}{7{,}333} - \frac{12 \times 1{,}297}{7{,}333} - \left(\frac{12\left[43.6 + 52.6 + 1.0(437)\right]}{9{,}446}\right)$$

$$= -0.002 \text{ ksi, within tensile limit -0.424 ksi} (\text{OK})$$

5.4.7.10.4 Fatigue Stress Limit

Fatigue of the reinforcement need not be checked for prestressed components designed to have extreme fiber tensile stress due to Strength III Limit State within the tensile stress limit specified in AASHTO-CA BDS-8 Table 5.9.2.3.2b-1, according to Article 5.5.3.1.

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Since the extreme fiber tensile stress is within specified limits, a fatigue stress check of the prestressing is not necessary for this example.

Concrete compressive stress limits for Fatigue I load combination, which consists of the Fatigue I live load and one-half the sum of the unfactored effective prestress and permanent loads, are:

$$\text{Compressive Limit} = 0.40f'_c = 0.40\,(5.0) = 2.00 \text{ ksi} \qquad \text{(AASHTO 5.5.3.1)}$$

The fatigue live load consists of the HL93 design truck with a constant axle spacing of 30.0 ft between 32.0-kip axles with dynamic allowance, as stated in California Amendment 3.6.1.4.1 (Caltrans, 2019). A Load Factor of 1.75 as specified in *AASHTO-CA BDS-8* Table 3.4.1-1 and the dynamic allowance (*IM*) of 15% (California Amendment Table 3.6.2.1-1) are used for Fatigue Limit State I. A structural analysis software is used to determine the following midspan moment demand.

M_f = unfactored Fatigue I live load moment with a dynamic allowance

$$M_f = M_{(LL+IM)Fatigue\ I}DFM = 973(0.161) = 130 \text{ kip-ft}$$

Note: (973 k-ft was obtained from software program)

The midspan top fiber girder stress from the Fatigue I load combination is calculated using the following equation:

$$f_{top} = \frac{P}{2A_g} - \frac{Pe_c}{2S_t} + \frac{M_{DC1}}{2S_t} + \left(\frac{0.5(M_{DC2} + M_{DW}) + \gamma_{LL}M_{LL+I}}{S_{tc}}\right)$$

$$= \frac{908}{2\times 795} - \frac{908\times 13.3}{2\times 6{,}892} + \frac{12\times 1{,}297}{2\times 6{,}892} + \left(\frac{12\left(0.5[43.6+52.6]+1.75\times 437\right)}{16{,}715}\right)$$

$$= 1.2 \text{ ksi} < 2.00 \text{ ksi}\,(\text{OK})$$

5.4.7.11 Design for Strength Limit State

5.4.7.11.1 Factored Moments

The factored moment (M_u) is the larger of Strength I and Strength II load combinations, per Article 3.4.1. Strength I uses the HL93 vehicular live load, whereas Strength II uses the California P-15 permit truck.

Midspan – Strength I Load Combination

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

(CA Amendment Table 3.4.1-1)

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where:

M_{DC1} = 1,297 kip-ft (Table 5.4-4)

M_{DC2} = 43.6 kip-ft (Table 5.4-4)

M_{DW} = 52.6 kip-ft (Table 5.4-4)

M_{LL+IM} = 437 kip-ft (Table 5.4-5)

$$M_u = 1.25(1297+43.6)+1.5(52.6)+1.75(437) = 2{,}519 \text{ kip-ft}$$

Midspan – Strength II Load Combination

$$M_u = 1.25\left[M_{DC1} + M_{DC2}\right] + 1.5M_{DW} + 1.35\left[M_{(LL+IM)P15}\right]$$ (CA Amendment Table 3.4.1-1)

M_{LL+IM} = 695 kip-ft (Table 5.4-7)

$$M_u = 1.25(1297+43.6)+1.5(52.6)+1.35(695) = 2{,}681 \text{ kip-ft}$$ (controls)

For the negative bending region, the moments at face-of-supports, which correspond to the bearing location, are used to calculate factored moments

Face of support – Strength I Load Combination at Bent 2

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

where:

M_{DC1} = 0 kip-ft (Table 5.4-4)

M_{DC2} = -67.6 kip-ft (Table 5.4-4)

M_{DW} = −81.6 kip-ft (Table 5.4-4)

M_{LL+IM} = −428 kip-ft (Table 5.4-5)

$$M_u = 1.25(-67.6) + 1.5(-81.6) + 1.75(-428) = -956 \text{ kip-ft}$$

Face of Support – Strength II Load Combination at Bent 2

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

M_{LL+IM} = −748 kip-ft (Table 5.4-7)

$$M_u = 1.25(-67.6) + 1.5(-81.6) + 1.35(-748) = -1{,}217 \text{ kip-ft}$$ (controls)

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5.4.7.11.2 Factored Moment Resistance

The factored moment resistance (M_r) is calculated at the midspan and the face-of-support, based on the requirements in AASHTO Article 5.6.3.

Midspan-Positive moment

At the midspan, the factored moment resistance is calculated as:

$$M_r = \phi M_n \quad \text{(AASHTO 5.6.3.2.1-1)}$$

where:

M_n = nominal flexural resistance assuming rectangular section conservatively ignoring the flexural resistance attributed to longitudinal reinforcement in the deck and the girder.

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

f_{ps} = stress in prestressing strand at ultimate

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

f_{pu} = nominal tensile stress of prestressing strand = 270 ksi

k = 0.28 for low-lax prestressing strand (AASHTO Table C5.6.3.1.1-1)

c = depth of compression at ultimate (AASHTO 5.6.3.1.1-4)

$$c = \frac{A_{ps} f_{pu}}{\alpha_1 f'_c \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}} \quad \text{(AASHTO 5.6.3.1.1-4)}$$

α_1 = stress block factor = 0.85 (AASHTO 5.6.2.2)

β_1 = stress block factor

For $f'_c \geq$ 0.4 ksi

$$\beta_1 = 0.85 - 0.05\left(f'_c - 4\right) = 0.85 - 0.05(5\text{-}4) = 0.8 > 0.65 \quad \text{(AASHTO 5.6.2.2)}$$

A_{ps} = area of prestressing steel = 5.21 in.^{2}

d_p = depth of prestressing strands for section = 39.0 - 2.7 = 36.3 in.

$$c = \frac{5.21(270.0)}{0.85(4.0)(0.80)(48.75) + 0.28(5.21)\frac{270.0}{36.3}} = 9.94 \text{ in.}$$

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Since the neutral axis depth (c) is less than the depth from the deck face to the void of the precast girder, the rectangular girder assumption is valid.

$$f_{ps} = 270\left(1 - 0.28\frac{9.94}{36.3}\right) = 249 \text{ ksi}$$

a = depth of the compression block

$$a = \beta_1 c = 7.96 \text{ in.}$$

$$M_n = 5.21(249)\left(36.3 - \frac{7.96}{2}\right) = 41{,}928 \text{ kip-in} = 3{,}494 \text{ kip-ft}$$

ϕ = resistance factor of precast girders (CA 5.5.4.2)

= 1.0 for $\varepsilon_t \geq .005$

= 0.75 for $\varepsilon_t \leq .002$

$$\phi = 0.75 + \frac{0.25(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \leq 1.0 \quad \text{(CA 5.5.4.2)}$$

ε_t = net tensile strain in extreme tension steel at nominal resistance (in./in.) (AASHTO 5.6.2.2)

$$\varepsilon_t = \frac{\varepsilon_{cu}}{c}(d_p - c)$$

ε_{cu} = concrete compression strain at nominal resistance = 0.003 in./in. (AASHTO 5.6.2.2)

ε_{tl} = net tensile strain – tension limit = 0.005 in./in. (AASHTO 5.6.2.2)

ε_{cl} = net tensile strain – compression limit = 0.002 in./in. (AASHTO 5.6.2.2)

$$\varepsilon_t = \frac{0.003}{9.94}(36.3 - 9.94) = 0.0080$$

Since the net tensile (ε_t) strain is greater than 0.005, the resistance factor (ϕ) is 1.0. Therefore,

$$M_r = 1.0(3{,}494) = 3{,}494 \text{ kip-ft} > M_u = 2{,}693 \text{ kip-ft (OK)}$$

Face of support-negative moment

The face of support is assumed to coincide with the centerline of the bearing for the non-composite girder and provides the maximum negative bending moments. At this location, the deck is in tension and the soffit of the precast girder is in compression. Therefore, the section is conventionally reinforced.

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$M_r = \phi M_n$ (AASHTO 5.6.3.2.1-1)

where the nominal flexural resistance (M_n) is calculated assuming the rectangular section conservatively ignoring the flexural resistance attributed to the longitudinal reinforcement in the girder.

$$M_n = A_s f_y \left(d - \frac{a}{2} \right)$$

f_y = 60 ksi

c = depth of compression at ultimate

$$c = \frac{A_s f_y}{\alpha_1 f_c' \beta_1 b} \quad \text{(AASHTO 5.6.3.1.1-4)}$$

α_1 = 0.85 for $f_c' \leq 10$ ksi (AASHTO 5.6.2.2)

β_1 = stress block factor

For $f_c' \geq$ 0.4 ksi

$\beta_1 = 0.85 - 0.05(f_c' - 4) = 0.85 - 0.05(5-4) = 0.8 > 0.65$ (AASHTO 5.6.2.2)

A_s = area of reinforcement steel (assume 9 #9 bars in the deck over each girder) = 9.00 in.^{2}

d = depth of the reinforcement, assumed centered in the deck to the soffit of the girder = 36.0 in.

$$c = \frac{9.00(60.0)}{0.85(5.0)(0.80)(48.0)} = 3.31 \text{ in.}$$

$$a = \beta_1 c = 0.80(3.31) = 2.65 \text{ in.}$$

$$M_n = 9.0(60)(36 - 2.65/2) = 18{,}725 \text{ kip-in.} = 1{,}560 \text{ kip-ft}$$

ϕ = resistance factor of non-prestressed sections = 0.9 for $\varepsilon_t \geq .005$ (CA 5.5.4.2)

$$\varepsilon_t = \frac{0.003}{3.31}(36.0 - 3.31) = 0.030 > 0.005 \text{ (tension controlled)}$$

$$M_r = 0.9(1{,}560) = 1{,}404 \text{ kip-ft} > M_u = 1{,}217 \text{ kip-ft (OK)}$$

5.4.7.11.3 Flexural Reinforcement Limits

Maximum Reinforcement

AASHTO-CA BDS-8 does not have a maximum limit for prestressed sections but requires

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a minimum net tensile strain of 0.004 for reinforced concrete sections.

Minimum Flexural Reinforcement

To prevent a brittle failure at the initial flexural cracking, Article 5.6.3.3 requires that all flexural components have sufficient prestressed and non-prestressed tensile reinforcement to develop a factored flexural resistance that is greater than a factored cracking moment, or 1.33 times the factored moments. Minimum reinforcement is evaluated at midspan and face of support.

Midspan

M_r ≥ the lesser of $1.33M_u$ and M_{cr}

where:

M_u = factored moment = 2,681 k-ft from Section 5.4.7.11.1

$$1.33M_u = 1.33(2,681) = 3,566 \text{ kip-ft}$$

M_{cr} = factored cracking moment

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

f_r = modulus of rupture of concrete

$$f_r = 0.24\lambda\sqrt{f'_c} = 0.24(1.0)\sqrt{5.0} = 0.537 \text{ ksi} \quad \text{(AASHTO 5.4.2.6)}$$

λ = 1.0 for normal weight concrete (AASHTO 5.4.2.8)

f_{cpe} = compressive stress in concrete due to effective prestress only

$$f_{cpe} = \frac{P_f}{A_{tf}} + \frac{P_f e_{tf}}{S_{Btf}} = \frac{908}{795} + \frac{908(13.3)}{7,333} = 2.79 \text{ ksi}$$

M_{dnc} = total unfactored dead load moment acting on the non-composite section (kip-ft)

$$M_{dnc} = 1,297 + 43 = 1,340 \text{ kip-ft}$$

S_c = section modulus for the extreme bottom fiber of the composite section where tensile stress is caused by externally applied loads = 9,446 in.^{3} (Table 5.4-12)

S_{nc} = section modulus for extreme the bottom fiber of the non-composite section where tensile stress is caused by externally applied loads = 7,333 in.^{3} (Table 5.4-11)

γ_1 = flexural cracking variability factor for other than PC segmental structures = 1.6

γ_2 = prestress variability factor for members with bonded tendons = 1.1

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γ_3 = ratio of specified minimum yield strength to ultimate tensile strength of reinforcement for prestressed concrete structures = 1.0

$$M_{cr} = 1.0\left[\left(1.6(0.537)+1.1(2.79)\right)(9{,}446) - 1{,}340(12)\left(\frac{9{,}446}{7{,}333} - 1\right)\right]$$

$$= 32{,}472 \text{ kip-in} = 2{,}706 \text{kip-ft}$$

$$\text{Min}(1.33M_u, M_{cr}) = 2{,}706 \text{ kip-ft} < M_r = 3{,}494 \text{kip-ft} \quad \text{(OK)}$$

Face of support - negative moment

$$M_{cr} = \frac{\gamma_3\,\gamma_1 f_r\, S_{ct}}{n}$$

γ_1 = 1.6

γ_3 = ratio of specified minimum yield strength to ultimate tensile strength of reinforcement for A706 Grade 60 reinforcement = 0.75

f_r = modulus of rupture of concrete

$$f_r = 0.24\lambda\sqrt{f'_c} = 0.24(1.0)\sqrt{4.0} = 0.48 \text{ ksi} \qquad \text{(AASHTO 5.4.2.6)}$$

S_c = section modulus for the extreme fiber of the composite section that is in tension occurs in the top of the deck (in.^{3})

$$S_c = I_c / (h - y_b) = (198{,}498)/(39.0 - 21.1) = 11{,}117 \text{ in.}^3$$

n = modular ratio between the deck and the girder = 0.929 (Section 5.4.7.8)

$$M_{cr} = (1.6)(0.75)(0.48)(11{,}117)/(0.929) = 6{,}892 \text{ kip-in.} = 574 \text{ kip-ft}$$

$$\text{Min}(1.33M_u, M_{cr}) = 574 \text{ kip-ft} < M_r = 1{,}404 \text{ kip-ft} \quad \text{(OK)}$$

5.4.7.12 Design for Shear

The shear design of precast box 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 and Article 5.7.3.4.2 of California Amendment (Caltrans, 2019). A design flow chart is provided in Figure CB5.2-5 of Appendix B5 of AASHTO-CA BDS-8.

Precast box girders are designed by comparing the factored shear forces (envelope values) and the factored shear resistances at tenth points along the member length and additional locations near supports. 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

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- 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 illustrates shear design only at the critical section.

5.4.7.12.1 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 allows 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. For this example, shear is conservatively assumed to be $H/2$ from the face of support, where H is the overall depth of the girder with the CIP concrete deck.

5.4.7.12.2 Factored Shear Force

The factored shear force V_u and corresponding factored moment at $H/2$, which is 1.63 feet from the face of the support, which coincides with the face of the column is the maximum of Strength I and Strength II load combinations.

Strength I

$$V_u = 1.25\left(V_{DC1} + V_{DC2}\right) + 1.5V_{DW} + 1.75V_{(LL+IM)HL93}$$

V_{DC1} = 51.7 kip (Table 5.4-4)

V_{DC2} = 4.4 kip (Table 5.4-4)

V_{DW} = 5.4 kip (Table 5.4-4)

$V_{(LL+IM)}$ = 60.3 k (HL93 live load with dynamic allowance) (Table 5.4-6)

$$V_u = 1.25\left(51.7 + 4.4\right) + 1.5\left(5.4\right) + 1.75\left(60.3\right) = 184 \text{ kips}$$

Strength II

$$V_u = 1.25\left(V_{DC1} + V_{DC2}\right) + 1.5V_{DW} + 1.35V_{(LL+IM)P-15}$$

$V_{(LL+IM)}$ = 112 k (P-15 live load with IM) (Table 5.4-8)

$$V_u = 1.25\left(51.7 + 4.4\right) + 1.5\left(5.4\right) + 1.35\left(112\right) = 229 \text{ kips (controls)}$$

Corresponding Strength II factored moment

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$$M_u = 1.25\left(M_{DC1} + M_{DC2}\right) + 1.5M_{DW} + 1.35M_{(LL+IM)P-15}$$

M_{DC1} = 85.7 kip-ft (Table 5.4-4)

M_{DC2} = -60.2 kip-ft (Table 5.4-4)

M_{DW} = -72.6 kip-ft (Table 5.4-4)

$M_{(LL+IM)}$ = -567 kip-ft (P-15 live load with IM) (Table 5.4-8)

$$M_u = 1.25\left(85.7–60.2\right) + 1.5\left(-72.6\right) + 1.35\left(-567\right) = -842 \text{ kip-ft}$$

5.4.7.12.3 Factored Shear Resistance

The factored shear resistance consists of concrete, reinforcing steel, and prestress contributions, as parts of the modified compression field theory.

$$V_r = \phi V_n$$

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

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

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

where:

b_v = effective web width taken as the minimum section width between the voids

= 2 x 5 = 10 in.

d_v = effective shear depth (in.)

$$d_v = d - a/2 \qquad \text{(AASHTO 5.7.2.8)}$$

where *a* is the equivalent stress block depth at the face of support.

$$d_v = d - a/2 = 36.0 - 2.65/2 = 34.7 \text{ in.}$$

$$0.72H = 0.72\left(39\right) = 28.1 \text{ in.}$$

$$0.9d_e = 0.9\left(36.0\right) = 32.4 \text{ in.}$$

$$d_v = 34.7 \text{ in.} > \max\left(0.72H,\ 0.9d_e\right) \quad \text{(OK)}$$

β = factor indicating the ability of diagonally cracked concrete to transmit tension and shear that is determined by the tables of AASHTO Appendix B-5.

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Based on the General Procedure of AASHTO Appendix B5, the value of β is based on the net longitudinal tensile strain, ε_x, at the mid-depth of the section for the normal case in which the code-minimum transverse reinforcement is provided (Figure 5.4-14). This is because such members have the capacity to redistribute shear stresses.

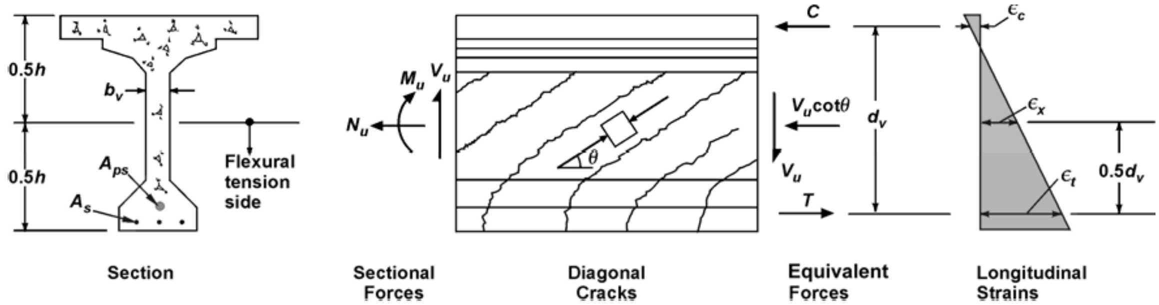

Figure 5.4-14 Shear Parameters for Section Containing at Least Minimum Amount of Transverse Reinforcement, V_p = 0, per Figure B5.2-1 (AASHTO, 2017)

Step 1 - Determine ε_x:

For the General Procedure, the longitudinal strain, ε_x, at the 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 cot(θ) may be taken equal to 1 (i.e., θ may be taken as 26.5°) initially during iterations for θ and β, and may also be assumed constant to avoid iterations, without significant loss of accuracy (AASHTO, 2017).

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

where:

M_u = factored moment at the section, not to be taken less than V_ud_v = 842 kip-ft

V_u = factored shear force = 229 kip

$V_ud_v = 229(34.7)/12 = 662 \text{ kip-ft} < M_u$

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

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V_p = component of prestressing force in the direction of the shear force: positive if resisting the applied shear = 0 kip (horizontal tendons)

θ = angle of inclination of diagonal compressive stresses = 26.5° initially assumed, based on taking 0.5 (cot θ) = 1

A_{ps} = area of prestressing strands on flexural tension side at section = 0.0 in.^{2}

A_s = area of non-prestressed steel on flexural tension side of member = 9.0 in.^{2}

$$\varepsilon_x = \frac{\left(\frac{12 \times 842}{34.7} + 229\right)}{2(29{,}000)(9.00)} = 0.997\ x\ 10^{-3}$$

Step 2 - Determine β and θ:

For sections with the transverse reinforcement equal to or larger than the minimum transverse reinforcement, the values 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 5.4-13. To use this, the ratio (v_u / f'_c) is required in addition to ε_x.

Using ϕ = 0.9 for shear,

$$v_u = \frac{\left|V_u - \phi V_p\right|}{\phi b_v d_v} \qquad \text{(AASHTO 5.7.2.8-1)}$$

$$v_u = \frac{\left|229 - 0\right|}{0.9(10.0)(34.7)} = 0.733 \text{ ksi}$$

$$\frac{v_u}{f'_c} = \frac{0.733}{5.0} = 0.147$$

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Table 5.4-13 Values of θ and β for Sections with Transverse Reinforcement (Table B5.2-1 AASHTO, 2017)

$\frac{v_u}{f'_c}$	$\varepsilon_x \times 1{,}000$								
	≤−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 4.10	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 2.50	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 2.60	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 2.40	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 1.70	34.3 1.58	35.8 1.50

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 the value of β in the column to the right (larger ε_x) of the computed value in the table.

For this design example, the first iteration yields:

$\theta = 37.3^\circ$

$\beta = 2.08$

The angle θ was initially assumed to be 26.5°, significantly less than 37.3°. Therefore, another iteration is performed using the angle of 37.3°.

$$\varepsilon_x = \frac{\left(\frac{12 \times 842}{34.7} + (0.5)\ 229 \cot 37.3\right)}{2(29{,}000)(9)} = 0.846\ x10^{-3}$$

From Table 5.4-13, Iteration 2 yields the same values for θ and β. Therefore, no further iteration is required, and the following values are used in the design at this section:

$\theta = 37.3^\circ$

$\beta = 2.08$

Step 3 - Compute concrete contribution to the shear resistance, V_c :

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

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$$V_c = 0.0316(1.0)(2.08)\sqrt{5.0}(10.0)(34.7) = 51.0 \text{ kip}$$

Step 4 – Determine required stirrup spacing, *s*

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

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

Solving this equation for V_s leads to:

$$V_s = \frac{V_u}{f} - V_c - V_p = \frac{229}{0.9} - 51.0 - 0 = 203 \text{ kip}$$

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

A_v = area of shear reinforcement within a distance *s* (in.^{2})

θ = angle of inclination of diagonal compressive stresses = 37.3 degrees

α = angle of inclination of transverse reinforcement to the longitudinal axis

= 90° for vertical stirrups

f_y = yield strength of transverse reinforcement (ksi)

$$\frac{A_v}{s} = \frac{203}{60(34.7)(\cot 37.3^\circ)\sin 90^\circ} = 0.074 \frac{\text{in.}^2}{\text{in.}}$$

Using #5 two-leg stirrups for transverse reinforcement,

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

Spacing, *s*:

$$s = \frac{0.62}{0.074} = 8.4 \text{ in.}$$

Use a spacing of 8 inches on the center (*s* = 8 in.) near supports. A larger spacing, up to the maximum permitted by AASHTO-CA BDS-8, should be selected beyond the critical

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section at the discretion of the designer.

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

$$V_s = \frac{(0.62)(60)(34.7)(\cot 37.3°)}{8} = 212 \text{ kip}$$

$$V_n = 51 + 212 = 263 \text{ kip}$$

$$V_r = 0.9(263) = 237 \text{ kip} > V_u = 229 \text{ kip} \quad (\text{OK})$$

5.4.7.12.4 Maximum Spacing of 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 the shear stress, v_u. From the previous calculation:

$$v_u = \frac{\left|V_u - \phi V_p\right|}{\phi b_v d_v} = 0.733 \text{ ksi}$$

If $v_u \ge 0.125 f'_c$, then $s_{max} = 0.4 d_v \le 12.0$ in.

$$0.125 f'_c = 0.125(5) = 0.625 \text{ ksi} < v_u$$

$$\because 0.4 d_v = 0.4(34.7) = 13.8 \text{ in.} > 12.0 \text{ in.}$$

$$\therefore s_{max} = 12.0 \text{ in.}$$

Therefore use 8-inch stirrup spacing near the supports with a maximum spacing of 12-inches near the midspan.

Note that tighter spacing per AASHTO Eq. 5.7.2.6-2 applies for cases in which:

$$v_u \ge 0.125 f'_c$$

5.4.7.12.5 Minimum Transverse Reinforcement

The area of transverse reinforcement, A_v, shall satisfy the minimum transverse reinforcement requirement as specified in Article 5.7.2.5 as follows:

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

For s = 12.0 in. maximum, as provided:

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$$A_v = 0.62\text{in.}^2 \geq 0.0316(1.0)\sqrt{5}\frac{10.0(12.0)}{60} = 0.14\text{in.}^2$$

Therefore, two legs of #5 stirrups at 12.0 inches on the center satisfy the minimum transverse reinforcement requirement.

5.4.7.12.6 Maximum Nominal Shear Resistance

To ensure that the web concrete will not crush prior to the yielding of the transverse reinforcement, Article 5.7.3.3 requires that the nominal shear resistance, V_n, be limited to:

$$V_n \leq 0.25 f_c' b_v d_v + V_p = 0.25(5)(10)(34.7) + 0 = 434 \text{ kips} \qquad \text{(AASHTO 5.7.3.3-2)}$$

which is greater than the nominal shear resistance of 263 kip. Therefore, the requirement is satisfied.

5.4.7.13 Check Longitudinal Reinforcement

The longitudinal reinforcement (including both prestressed and non-prestressed reinforcement on the flexural tension side) must be proportioned to satisfy flexure-shear interaction requirements. Since the girder supports are direct, the flexure shear interaction need not be checked closer than a distance d_v from the face of the support. Conservatively, the shear and flexure-shear interaction are evaluated at *H/*2 from the face of the support in this example.

The following equation is used to evaluate the longitudinal reinforcement requirement for the flexure-shear interaction:

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

Two conditions are checked in the negative bending region at *H*/2. These include the maximum moment and the corresponding shear, and the maximum shear and the corresponding moment.

Condition 1 – the maximum moment and the corresponding shear

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

M_{DC1} = 85.7 kip-ft (Table 5.4-4)

M_{DC2} = – 60.2 kip-ft (Table 5.4-4)

M_{DW} = –72.6 kip-ft (Table 5.4-4)

$M_{(LL+IM)}$ = – 678 kip-ft (P-15 live load) (Table 5.4-7)

$$M_u = 1.25(85.7 - 60.2) + 1.5(-72.6) + 1.35(-678) = -\ 992 \text{ kip-ft}$$

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The factored corresponding shear (V_u) at $H/2$ is

$$V_u = 1.25(V_{DC1} + V_{DC2}) + 1.5V_{DW} + 1.35V_{(LL+IM)P15}$$

V_{DC1} = 51.7 kip (Table 5.4-4)

V_{DC2} = 4.4 kip (Table 5.4-4)

V_{DW} = 5.4 kip (Table 5.4-4)

$V_{(LL+IM)}$ = 75.9 kip (P-15 live load) (From analysis software)

$$V_u = \ = 1.25(51.7 + 4.4) + 1.5(5.4) + 1.35(75.9) = 182 \text{ kip}$$

$$V_s = \frac{V_u}{f} - V_c = \frac{182}{0.9} - 51 = 151 \text{ kip}$$

V_p = 0 kip

N_u = 0 kip

D_v = 34.7 in.

θ = 37.3°

The determination of longitudinal reinforcement for the flexure-shear interaction at $H/2$ from the face of the support is calculated as follows.

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

$$= \frac{|992(12)|}{34.7(1.0)} + 0.5\frac{0.0}{1.0} + \left(\left|\frac{182}{0.9} - 0\right| - 0.5(151)\right)\cot 37.3°$$

$$= 509 \text{ kip} < A_s f_y = 540 \text{ kip (OK)}$$

Condition 2 – the maximum shear and the corresponding moment

M_u = 842 kip-ft (Section 5.4.7.12.3)

V_u = 229 kip (Section 5.4.7.12.3)

V_s = 212 kip (Section 5.4.7.12.3)

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

$$= \frac{|842(12)|}{34.7(1.0)} + 0.5\frac{0.0}{1.0} + \left(\left|\frac{229}{0.9} - 0\right| - 0.5(212)\right)\cot 37.3°$$

$$= 486 \text{ kip} < A_s f_y = 540 \text{ kip (OK)}$$

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5.4.7.14 Design Pretensioned Anchorage Zone Reinforcement

Article 5.9.4.4 requires the following horizontal reinforcement to be provided within the distance *h*/4 from the end of the girder to provide the splitting resistance to bursting stresses.

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

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 the end of girder (in.^{2})

h = overall dimension (width) of the precast box 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

$$P_r = 0.04(P_i) = 0.04(0.75)(270)(5.21) = 42.2 \text{ kip}$$

$$A_s = \frac{42.2}{20} = 2.11 \text{ in.}^2$$

Considering #5 stirrups on the exterior surfaces of the girder:

$$\text{Number of bars required} = \frac{2.11}{0.31(2)} = 3.4$$

Therefore, use four #5 stirrups within h/4 (33/4 = 8.3 in.) from the end of the girder at a 2.0-inch spacing.

In addition, Article 5.9.4.4.2 requires reinforcement to 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 with spacing not to exceed 6 inches and shaped to enclose the strands.

Use #5 stirrups at 6 inches on the center, to comply with the splitting requirement, over a minimum distance of 1.5*d* = 1.5(33) = 49.5 inches from the end of the girder to confine the prestressing strands. Use 3-spaces at 3.5 inches followed by 7 spaces at 6 inches.

5.4.7.15 Estimate Deflection and Camber

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

- instantaneous girder deflections due to the topping slab, the overlay, and barrier rail, which is used by the contractor to set screed rail elevations for the deck pour.

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- long-term deflections to determine if the girder will have a sag or an arch.

Refer to Chapter 5.3 for the applicable deflection equations and associated illustrations. The 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 gross moment of inertia. Long-term deflections consist of long-term deflections at the erection and long-term deflections at the final stage (may be assumed to be approximately 20 years).

5.4.7.15.1 Concrete Deck, Barrier Rail, and Sidewalk Deflections

In this section, the deflections due to the wet concrete weight of the deck, the barriers, and the sidewalks are calculated for the contract plans. The girder initially deflects upward upon the transfer of the prestress creating the camber. This camber continues to grow due the to creep of the concrete. The placement of the concrete deck creates an instantaneous downward deflection. After this downward deflection, the camber growth essentially stops. Therefore, elastic deflections associated with the concrete deck, the sidewalk, and the barrier are shown on the contract plans. These deflections are used to adjust screed rail elevations for the concrete deck pour to attain the desired profile.

The wet concrete weight of the concrete deck acts on the non-composite simply supported girder. The deflection of the girder at the mid-span is calculated as:

$$\Delta_t = \frac{5 w_s L^4}{384 E_c I}$$

w_s = wet concrete weight of the CIP deck = 0.305 kip/ft

L = length of the girder = 97 ft

E_c = final elastic modulus of the girder = 4,291 ksi

I = gross moment of inertia of the girder = 111,829 in.^{4}

$$\Delta_t = \frac{5\left(\frac{0.305}{12}\right)(97 \times 12)^4}{384(4,291)111,829} = 1.27 \text{ in.}$$

After the deck concrete hardens, the bridge becomes a composite continuous structure. The deflections due to the weight of the sidewalk and barrier on the composite structure are calculated using the structural analysis software.

$$\Delta_b = 0.06 \text{ in.} (\text{barrier and sidewalk})$$

The contractor will set the screed rails 1.33 inches higher than the profile grade at the midspan of Span 2 to achieve the desired profiled grade at the end of construction.

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5.4.7.15.2 Long Term Girder Deflections

The long-term deflections can be estimated using creep multipliers and the elastic deflections superimposed at milestone events. The following are the remaining elastic components of the deflection.

Girder prestress

$$\Delta_p = \frac{P_i e_c L^2}{8E_{ci} I}$$

P_i = initial prestress force = 1,055 kip

e_c = initial strand eccentricity = 13.8 in.

L = length of the girder = 97 ft

E_{ci} = elastic modulus of the girder = 3,987 ksi

I = gross moment of inertia of the non-composite girder = 111,829 in.^{4}

$$\Delta_p = \frac{1{,}055\left(13.8\right)\left(97\times12\right)^2}{8\times3{,}987\times111{,}829} = 5.53 \text{ in.}$$

Girder Self Weight

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

w_g = girder self-weight = 0.798 kip/ft

L = length of the girder = 97 ft

E_{ci} = initial elastic modulus of the girder = 3,987 ksi

I = gross moment of inertia of the non-composite girder = 111,829 in.^{4}

$$\Delta_g = \frac{5\left(\frac{0.798}{12}\right)\left(97\times12\right)^4}{384\left(3{,}987\right)111{,}829} = 3.57 \text{ in.}$$

The permanent midspan deflections are summarized in Table 5.4-13, which include multipliers to account for creep between the transfer and the erection. The respective multipliers at the erection and the completion of construction are assumed equal due to the short duration and the minimal creep deflection anticipated between these two milestones. A multiplier of 1.0 is used for the deck and the barrier for the same reason. The multipliers in Chapter 5.3 are not recommended for estimating long-term deflections.

A sample calculation of the deflections at the completion of construction is shown for the

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prestressing as follows:

$$\Delta_{p(completion)} = \Delta_{p(elastic)} \text{ x multiplier} = 5.53\ (1.80) = 9.95 \text{ in.}$$

Deflections due to the application of a future wearing surface (*DW*) are calculated using the structural analysis software. These deflections are not used for setting the screed rails because the overlay could be applied after the bridge has been in service for many years. Also, creep deflections are not anticipated to be significant since the concrete will be relatively mature during the overlay placement.

$$\Delta_{DW} = 0.07 \text{ in.}$$

Table 5.4-14 Tabulated Midspan Deflections (in.) (up is positive)

Item	Elastic	Multiplier*	Erection	Multiplier	Completion of Construction
Prestress	5.53	1.80	9.95	1.80	9.95
Girder Self Weight	-3.57	1.85	-6.60	1.85	-6.60
CIP Concrete Deck	-1.27			1.00	-1.27
Barrier and Sidewalk	-0.06			1.00	-0.06
Total			3.35		2.02
*Multiplier, per PCI (2014)					

As shown in Table 5.4-14, the multipliers indicate that the girder will have a net camber at the bottom of the girder of 3.35 inches upward at erection. After placement of the concrete deck, the barrier, and the sidewalk, the driving surface should match the desired profile grade of the roadway. At the completion of construction, a camber of 2.02 inches is anticipated along the soffit of the girder. Similarly, the deck thickness is estimated to be 2.02 inches thicker at the supports than at the midspan if the profile grade is flat.

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NOTATION

a	=	depth of equivalent rectangular compression stress block (in.)
A	=	area of stringer, beam, or component (in.^{2})
A_c	=	concrete area of composite section (in.^{2})
A_c	=	concrete area of transformed section (in.^{2})
A_{cv}	=	area of concrete engaged in interface shear transfer (in.^{2})
A_{ct}	=	area of concrete on the flexural tension side of member (in.^{2})
A_g	=	gross area of beam section (in.^{2})
A_i	=	area of an individual component (in.^{2})
A_0	=	area enclosed by the centerline of elements (in.^{2})
A_{ps}	=	area of prestressing steel (in.^{2})
A_{rect}	=	area of a rectangle (in.^{2})
A_s	=	area of non-prestressed tension reinforcement (in.^{2})
A'_s	=	area of compression reinforcement (in.^{2})
A_{tf}	=	area of the transformed section, at final (in.^{2})
A_{ti}	=	area of the transformed section, at initial (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})
A_{void}	=	area of void (in.^{2})
ADL	=	added dead load (kip)
b	=	width of the compression face of a member (in.)
b_{eff}	=	effective flange width (in.)
b_L	=	distance from end of the beam 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.); cohesion factor from AASHTO Art 5.8.4.3 (ksi)
cg	=	center of gravity
CGC	=	center of gravity of the concrete section

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CGS	=	center of gravity of the strands
D	=	structure depth or height of standard shape of PC beam (ft)
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.)
DFDL	=	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 centroid of the nonprestressed tensile reinforcement (in.)
D_s	=	superstructure depth (ft)
d_t	=	distance from extreme compression fiber to the centroid of tensile reinforcement (in.)
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 (ft); also, the base of Napierian logarithms
e'	=	difference between eccentricity of prestressing steel at midspan and the end (in.)
E_B, E_c	=	modulus of elasticity of beam material (ksi)
e_c	=	eccentricity of strands measured from the center of gravity of beam at midspan (in.)
E_{ci}	=	modulus of elasticity of concrete at the initial time (ksi)
E_{ct}	=	modulus of elasticity of concrete at transfer or time of load application (ksi)
E_{cu}	=	failure strain of concrete in compression (in./in.)
E_D	=	modulus of elasticity of deck material (ksi)
e_g	=	distance between centers of gravity of beam and deck (in.)
e_m	=	eccentricity at midspan (in.)
E_p, E_{ps}	=	modulus of elasticity of prestressing tendons (ksi)

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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_{bot} = concrete stress at bottom of the precast beam (ksi)

f'_c = specified compressive strength of concrete used in design (ksi)

f'_{ci} = specified compressive strength of concrete at the time of initial loading or prestressing (ksi); nominal concrete strength at the time of application of tendon force (ksi)

f_{cgp} = concrete stress at the center of gravity of prestressing tendons 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 the extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

f_g = stress in the member from the 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_{px} = reduced prestressing steel stress in transfer length (ksi)

f_r = modulus of rupture of concrete (ksi)

f_s = stress in mild tension reinforcement at nominal flexural resistance (ksi)

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

F_T = required tension capacity provided by reinforcement (ksi)

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f_{tg}	=	concrete stress at top of precast beam (ksi)
f_{tgf}	=	concrete stress at top of the precast beam due to Fatigue I combination (ksi)
f_{top}	=	concrete stress at top of precast beam (ksi)
f_y	=	yield strength of mild steel (ksi)
f'_y	=	specified minimum yield strength of compression reinforcement (ksi)
H	=	average annual ambient mean relative humidity (percent)
h	=	web dimension of PC beam (in.)
h_c	=	overall depth of composite section (in.)
h_{girder}	=	overall depth of beam (in.)
I	=	initial gross (non-transformed) moment of inertia of the beam (in.^{4})
I_c	=	moment of inertia of composite section about a centroidal axis, neglecting reinforcement (in.^{4})
I_{cg}, I_g	=	moment of inertia of the beam 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 beam (in.^{4})
I_o	=	moment of inertia of individual component (in.^{4})
I_{rect}	=	moment of inertia of rectangle (in.^{4})
I_{tf}	=	moment of inertia of concrete section at the final stage, transformed (in.^{4})
I_{ti}	=	moment of inertia of concrete section at the initial stage, transformed (in.^{4})
I_{void}	=	moment of inertia of void (in.^{4})
J	=	St. Venant torsional constant (in.^{4})
K_1	=	fraction of concrete strength available to resist shear
K_2	=	limiting interface shear resistance
K_g	=	longitudinal stiffness parameter (in.^{4})
K_L	=	span to depth correction factor (in.^{4})
K_S	=	skew correction factor (in.^{4})
L	=	span length or beam length (ft)
LL	=	live load (kip)
M_{cr}	=	cracking moment (kip-in.)

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M_{ADL} = moment due to added dead loads (kip-ft)

M_{DC1} = moment due to self-weight of the beam (kip-ft)

M_{DC2} = moment due to self-weight of the deck (kip-ft)

M_{DC3} = moment due to self-weight of barrier (kip-ft)

M_{DW} = moment due to self-weight of overlay (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_f = midspan moment due to fatigue truck (kip-ft)

M_g = midspan moment due to self-weight of the beam (kip-ft)

M_{HL93} = moment due to enveloped HL93 trucks (kip-ft)

M_{LL} = moment due to live loads (kip-ft)

M_n = nominal flexure resistance (kip-ft)

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-ft)

M_{slab} = moment due to weight of deck slab (kip-ft)

M_u = controlling factored moment demand (kip-ft)

M_x = moment at location x (kip-ft)

n = modular ratio between beam and deck

N_b = number of beams, stringers, or beams

N_u = factored axial force, taken as positive if tensile and negative is compressive (kip)

P, P_e = effective force in prestressing strands after all losses (kip)

P = required transverse post-tensioning (kip/ft)

P_c = permanent compressive force (kip)

P_{dia} = transverse diaphragm compressive force (kip)

P_f = force in prestressing strands after losses (kip)

P_{fg} = effective force in prestressing strands after all losses for gross section design (kip)

P_i = force in prestressing strands after elastic shortening loss (kip)

P_j = force in prestressing strands before losses (kip)

P_r = factored bursting resistance of anchorages (kip)

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S	=	spacing of beams or webs (ft)
s	=	spacing of transverse reinforcement (in.)
S_b	=	section modulus for the bottom extreme fiber of the beam 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 or precast beam - transformed, at service stage (in.^{3})
S_{Bti}	=	section modulus for the bottom extreme fiber of the composite section or precast beam - transformed, at the initial 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 beam level, at service, transformed (in.^{3})
S_{Ttf}	=	section modulus for the top extreme fiber of the composite sections at top of beam level or precast beam, at service, transformed (in.^{3})
S_{Tti}	=	section modulus for the top extreme fiber precast beam, at initial, transformed (in.^{3})
T	=	tensile stress in concrete (ksi)
t	=	thickness of plate like element (in.)
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—the weight of barrier (kip/ft)

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w_c = unit weight of concrete (kcf)

w_{DW} = uniform dead load—the weight of overlay (kip/ft)

w_{fw} = uniform dead load—the weight of future wearing surface (kip/ft)

w_g = uniform dead load—the weight of beam (kip/ft)

w_h = uniform dead load—weight of haunch (kip/ft)

w_s = uniform dead load—weight of deck slab (kip/ft)

V = volume of the beam (in.^{3})

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 (kip)

V_u = factored shear force (kip)

v_u = average factored shear stress on the concrete (ksi)

V_{ui} = factored interface shear resistance (kip/in)

x = distance from the left end of the beam (ft)

x_{pA} = influence the length of anchor set (ft)

Y = distance from the neutral axis to a point on an individual component (in.)

y_b = distance from the neutral axis to the extreme bottom fiber of PC beam (in.)

Y_{BC} = distance from the centroid to extreme bottom fiber of composite section (in.)

Y_{BCt} = distance from the centroid to extreme bottom fiber of composite section, at service, transformed (in.)

Y_{Btf} = distance from the centroid to extreme bottom fiber of PC beam, at service, transformed (in.)

Y_{Bti} = distance from the centroid to extreme bottom fiber of PC beam, 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.)

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y_t	=	distance from the neutral axis to the extreme top fiber of PC beam (in.)
Y_{TC}	=	distance from the centroid to extreme top fiber of composite section (in.)
Y_{tg}	=	distance from the centroid of the composite section to the extreme top fiber of the PC beam (in.)
Y_{TGCt}	=	distance from the centroid of the composite section to the extreme top fiber of the PC beam (in.)
Y_{Tti}	=	distance neutral axis to the extreme top fiber of the PC beam, transformed (in.)
α	=	angle of inclination of transverse reinforcement to the longitudinal axis (°) total angular change of prestressing steel path from jacking end to a point under investigation (rad)
α_1	=	stress block factor
β	=	factor relating to the effect of longitudinal strain on the shear capacity of concrete, as indicated by the ability of diagonally cracked concrete to transmit tension
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_{Aset}	=	anchor set length (in.)
Δ_{br}	=	deflection due to barrier weight (in.)
Δ_g	=	camber at midspan at erection due to beam self-weight (in.)
$\Delta_{g,erect}$	=	camber at midspan at erection due to long-term effects of prestressing force and beam self-weight (in.)
Δ_{ES}	=	change in length due to elastic shortening (in.)
Δf_{pA}	=	losses due to anchorage set (ksi)
Δf_{pCD}	=	prestress loss due to creep of beam concrete between the time of deck placement and final time (ksi)
Δf_{pCR}	=	prestress loss due to creep of beam concrete between transfer and deck placement (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_{pF}	=	losses due to friction (ksi)
Δf_{pLT}	=	losses due to long-term shrinkage and creep of concrete and relaxation of prestressing steel (ksi)

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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 the strand (ksi)
Δf_{pR1}	=	prestress loss due to relaxation of prestressing strands between the time of transfer and deck placement (ksi)
Δf_{pR2}	=	prestress loss due to relaxation of prestressing strands in the composite section between the time of deck placement and final time (ksi)
Δf_{pSD}	=	prestress loss due to shrinkage of beam concrete between the time of deck placement and final time (ksi)
Δf_{pSR}	=	prestress loss due to shrinkage of beam concrete between transfer and deck placement (ksi)
Δf_{pSS}	=	prestress gain due to shrinkage of the deck in composite section (ksi)
Δf_{pT}	=	total loss (ksi)
Δ_{fw}	=	deflection due to future wearing surface (in.)
Δ_g	=	deflection due to beam self-weight (in.)
Δ_{LL}	=	deflection due to lane live load (in.)
Δ_{LT}	=	deflection due to truck live load (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 or skew angle (degree)
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 a relative humidity of ambient air
γ_{st}	=	correction factor for specified concrete strength time at of prestress transfer to the concrete member
ϕ	=	resistance factor
μ	=	coefficient of friction

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