5.3 PRECAST PRESTRESSED GIRDERS

5.3.1 GENERAL

This memo provides general design guidance for precast prestressed concrete girders in conformance to the AASHTO-CA BDS-8 and provides detailed commentary and explanations to reflect common Caltrans practices.

5.3.2 INTRODUCTION

A precast prestressed concrete girder bridge can be an economical and preferred solution when the bridge project faces considerations such as, but not limited to, the following:

- Falsework restrictions
- Limited construction time
- Limited vertical clearance
- Minimal traffic disruptions
- Environmental impact requirements
- Complex construction staging
- Preservation of existing roadway alignment
- Maintaining existing traffic
- Provisions for future deck replacement

Furthermore, precast girders can be more effective and economical when the girder quantity is large and details are repeatable. Project Engineers are encouraged to consider precast prestressed concrete girder superstructures as an alternative during the planning phase.

5.3.3 DESIGN PROCEDURE AND CONSIDERATIONS

In general, the design of precast prestressed concrete girders includes the following: establishing bridge geometry, selecting girder section and materials, calculating loads and load effects, determining prestressing force and losses, performing flexural design, performing shear design, checking anchorage zone and estimating camber and deflection.

For prestressed concrete girders satisfying the Service III limit state, the fatigue limit state need not be checked.

AASHTO-CA BDS-8 Article 3.5.1 requires that the weight of the deck between the edges of girder flanges be increased by 10% to account for any stay-in-place metal deck forms (SIPMF) used during construction. AASHTO-CA BDS-8 Article 9.7.4.2 prohibits the use of SIPMF in freeze-thaw areas and marine environments.

During the design phase, the designer is responsible for ensuring that the precast girder selected can be constructed in the precast plant and delivered to and erected at the job site. During the construction phase, the Contractor is responsible for the design of the precast girders for storage, handling, shipping, and erecting as well as for the girder stability check. If the Contractor proposes design changes based on the aforementioned considerations through the shop drawing submittal process, the designer shall verify that the modifications do not affect the original design calculations. Otherwise, the changes should be considered as the Contractor's Change Order.

5.3.4 TYPICAL PRECAST PRESTRESSED GIRDER SECTIONS AND SPAN LENGTHS

Caltrans has standard precast prestressed concrete girder sections, as shown in 5.3.4.1. Several precast prestressed girder standard details sheets and corresponding user guides are provided at the Caltrans Bridge Standard Details website. Because many precast girder forms are premade and standard, the precast industry needs to be consulted when any modifications to a standard shape are being considered. The modification may not be feasible or economically viable.

Typical precast prestressed girders are cast on a flat surface, and vertical clearance calculations need to take this into consideration.

5.3.4.1 Standard Girder Sections

California Standard “I” Girder

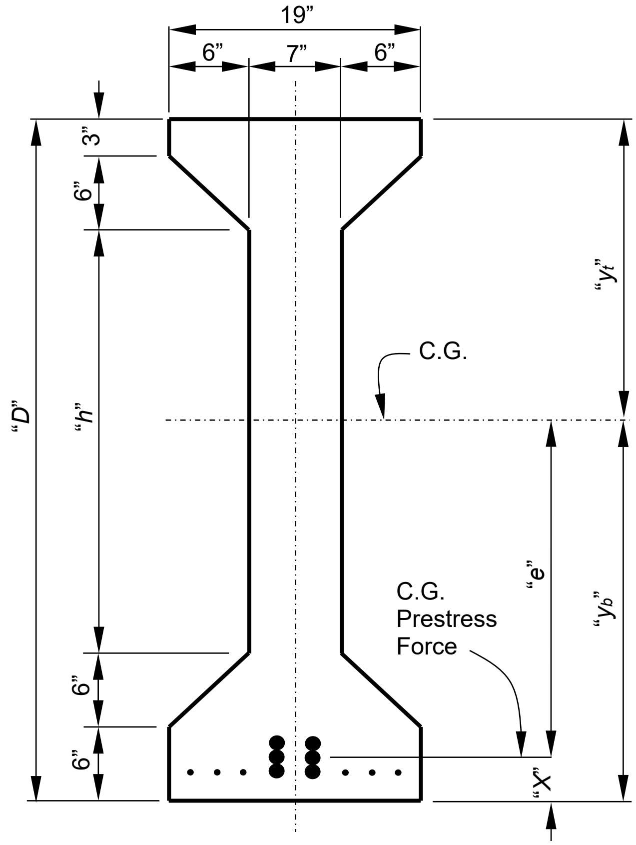


Figure 5.3.4.1-1 California Standard “I” Girder Typical Section

Table 5.3.4.1-1 Section Properties of California Standard “I” Girder

Girder Series	“*D*” (ft-in)	“*h*” (in)	Gross Area (in^2)	I_{cg} (in^4)	y_b (in)	y_t (in)	S_b (in^3)	S_t (in^3)	*r* (in)	Weight (lbs/ft)
CA I36	3’-0’’	15	432	63,000	17.1	18.9	3,684.2	3,333.3	12.1	450
CA I42	3’-6’’	21	474	95,400	20.0	20.0	4,770.0	4,336.4	14.2	494
CA I48	4’-0’’	27	516	136,400	22.8	25.2	5,982.5	5,412.7	16.3	538
CA I54	4’-6’’	33	558	186,400	25.7	28.3	7,264.6	6,597.2	18.3	581
CA I60	5’-0’’	39	600	246,900	28.6	31.4	8,632.9	7,863.1	20.3	625
CA I66	5’-6’’	45	642	318,000	31.6	34.4	10,063.3	9,244.2	22.3	669

California Standard "Bulb-Tee" Girder

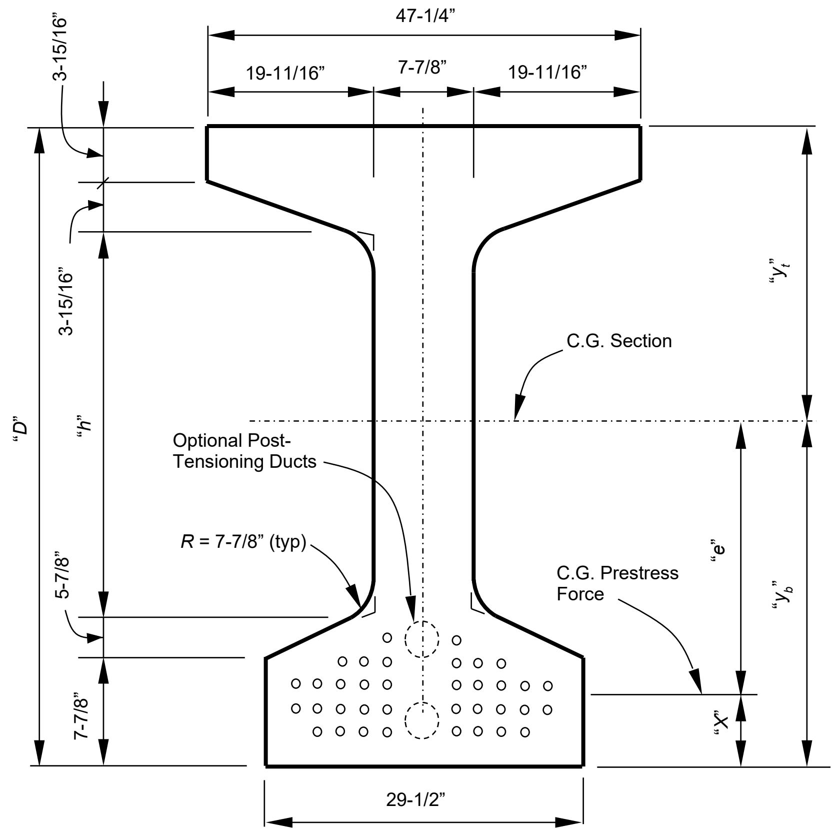


Figure 5.3.4.1-2 California Standard "Bulb-Tee" Girder Typical Section

Table 5.3.4.1-2 Section Properties of California Standard "Bulb-Tee" Girder

Girder Series	"*D*" (ft-in)	"*h*" (in)	Gross Area (in^2)	I_{cg} (in^4)	y_b (in)	y_t (in)	S_b (in^3)	S_t (in^3)	*r* (in)	Weight (lbs/ft)
CA BT49	4'-1''	27.375	856	272,373	25.3	23.7	10,766	11,493	18	892
CA BT55	4'-7 1/8''	33.465	925	373,350	28.4	26.7	13,153	13,966	20	963
CA BT61	5'-1''	39.370	971	483,385	31.3	29.7	15,444	16,262	22	1,012
CA BT67	5'-6 7/8''	45.276	1,018	610,718	34.2	32.7	17,830	18,689	24	1,060
CA BT73	6'-0 7/8''	51.181	1,063	755,589	37.2	35.6	20,309	21,207	27	1,108
CA BT79	6'-6 3/4''	57.087	1,110	919,200	40.1	38.6	22,912	23,800	29	1,157
CA BT85	7'-0 5/8''	62.992	1,157	1,102,271	43.1	41.6	25,592	26,513	31	1,205

Note: The dimensions are converted from metric values.

California Standard Pretensioned “Wide-Flange” Girder

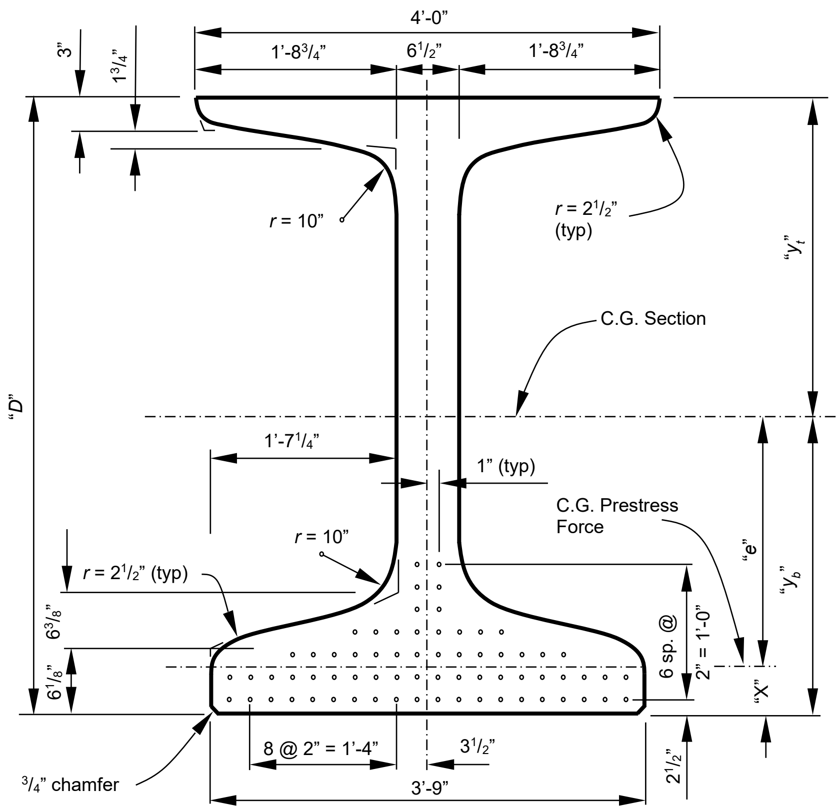


Figure 5.3.4.1-3 California Standard Pretensioned “Wide-Flange” Girder Typical Section

Table 5.3.4.1-3 Section Properties of California Standard Pretensioned “Wide-Flange” Girder

Girder Series	“*D*” (ft-in)	“*D*” (in)	Gross Area (in²)	I_{cg} (in⁴)	y_b (in)	y_t (in)	S_b (in³)	S_t (in³)	*r* (in)	Weight (lbs/ft)
CA WF48	4’-0”	48	882	274,880	20.69	27.31	13,290	10,070	17.65	920
CA WF54	4’-6”	54	921	369,100	23.23	30.77	15,890	12,000	20.02	960
CA WF60	5’-0”	60	960	479,620	25.82	34.18	18,580	14,030	22.35	1,000
CA WF66	5’-6”	66	999	607,130	28.44	37.56	21,350	16,160	24.65	1,040
CA WF72	6’-0”	72	1,038	752,430	31.08	40.92	24,210	18,390	26.92	1,080
CA WF78	6'-6"	78	1,075	913,490	33.77	44.23	27,050	20,650	29.15	1,120
CA WF84	7’-0”	84	1,116	1,099,400	36.45	47.55	30,160	23,120	31.39	1,160
CA WF90	7'-6"	90	1,153	1,297,900	39.19	50.81	33,120	25,550	33.55	1,200
CA WF96	8’-0”	96	1,194	1,526,600	41.91	54.09	36,430	28,220	35.76	1,240
CA WF102	8'-6"	102	1,231	1,764,700	44.68	57.32	39,500	30,790	37.86	1,280
CA WF108	9’-0”	108	1,272	2,039,200	47.42	60.58	43,000	33,660	40.04	1,330
CA WF114	9'-6"	114	1,309	2,319,500	50.23	63.77	46,180	36,370	42.09	1,360
CA WF120	10’-0”	120	1,350	2,643,200	52.99	67.01	49,880	39,450	44.25	1,410

California Standard Post-Tensioned “Wide-Flange” Girder

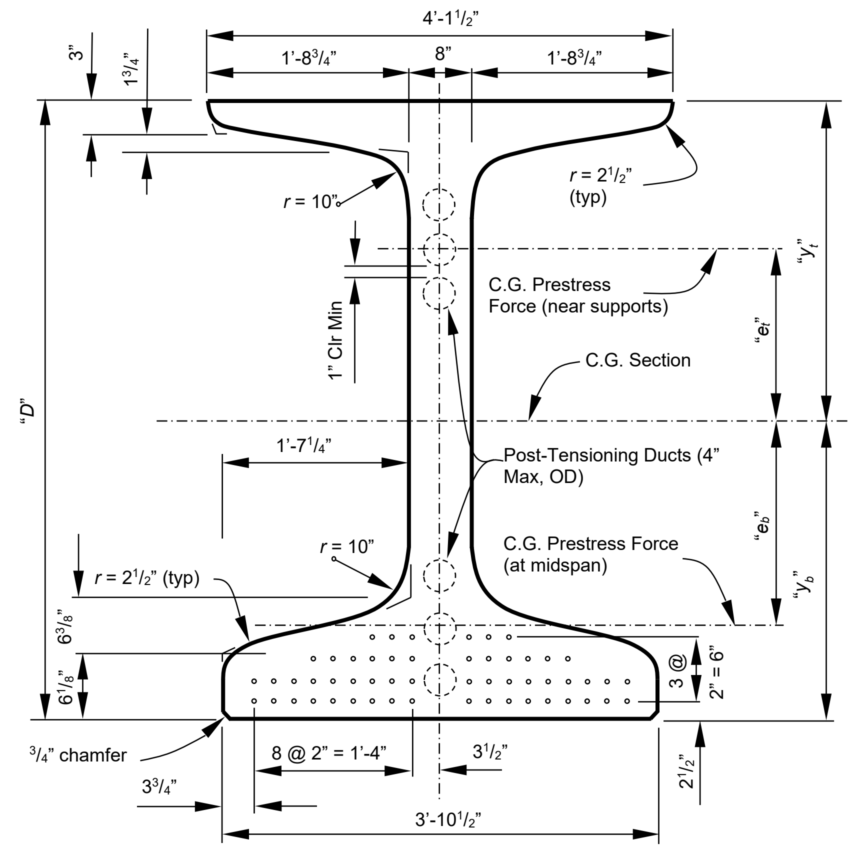


Figure 5.3.4.1-4 California Standard Post-Tensioned “Wide-Flange” Girder Typical Section

Table 5.3.4.1-4 Section Properties of California Standard Post-Tensioned “Wide-Flange” Girder

Girder Series	“*D*” (ft-in)	“*D*” (in)	Gross Area (in^2)	*I* (in^4)	“y_b” (in)	“y_t” (in)	S_b (in^3)	S_t (in^3)	*r* (in)	Weight (lbs/ft)
CA WF48PT	4’-0”	48	954	289,440	20.94	27.06	13,820	10,700	17.42	990
CA WF54PT	4’-6”	54	1,002	389,840	23.54	30.46	16,560	12,800	19.72	1,040
CA WF60PT	5’-0”	60	1,050	508,060	26.18	33.82	19,410	15,020	22.00	1,090
CA WF66PT	5’-6”	66	1,098	644,950	28.85	37.15	22,360	17,360	24.24	1,140
CA WF72PT	6’-0”	72	1,146	801,460	31.55	40.45	25,400	19,810	26.45	1,190
CA WF78PT	6'-6"	78	1,192	975,690	34.29	43.71	28,450	22,320	28.61	1,240
CA WF84PT	7’-0”	84	1,242	1,177,000	37.01	46.99	31,800	25,050	30.78	1,290
CA WF90PT	7'-6"	90	1,288	1,393,100	39.80	50.20	35,000	27,750	32.89	1,340
CA WF96PT	8’-0”	96	1,338	1,642,000	42.56	53.44	38,580	30,730	35.03	1,390
CA WF102PT	8'-6"	102	1,384	1,902,800	45.38	56.62	41,930	33,610	37.08	1,440
CA WF108PT	9’-0”	108	1,434	2,202,900	48.16	59.84	45,740	36,810	39.19	1,490
CA WF114PT	9'-6"	114	1,480	2,511,700	51.01	62.99	49,240	39,870	41.20	1,540
CA WF120PT	10’-0”	120	1,530	2,867,100	53.81	66.19	53,280	43,320	43.29	1,590

California Standard “Bath-Tub” Girder

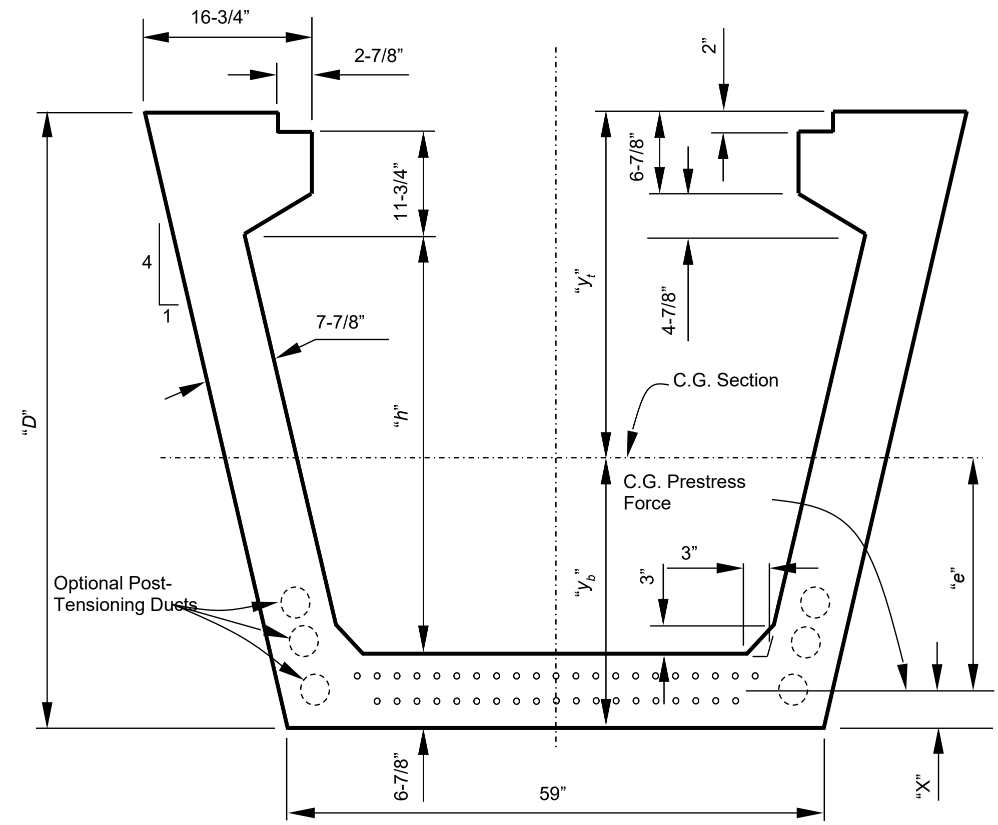


Figure 5.3.4.1-5 California Standard “Bath-Tub” Girder Typical Section

Table 5.3.4.1-5 Section Properties of California Standard “Bath-Tub” Girder

Girder Series	“*D*” (ft-in)	“*D*” (in)	Gross Area (in^2)	I_{cg} (in^4)	y_b (in)	y_t (in)	S_b (in^3)	S_t (in^3)	*r* (in)	Weight (lbs/ft)
CA TUB55	4’-7 1/8’’	36.4	1,339	460,081	24.1	31.0	19,095	14,830	19	1,395
CA TUB61	5’-1’’	42.3	1,435	604,231	26.9	34.1	22,471	17,702	21	1,495
CA TUB67	5’-6 7/8’’	48.2	1,531	773,128	29.7	37.2	26,010	20,780	22	1,595
CA TUB73	6’-0 7/8’’	54.1	1,627	968,692	32.6	40.3	29,752	24,052	24	1,695
CA TUB79	6’-6 3/4’’	60.0	1,723	1,192,606	35.4	43.3	33,695	27,513	26	1,795
CA TUB85	7’-0 5/8’’	65.9	1,819	1,446,551	38.3	46.4	37,801	31,190	28	1,895

Note: The dimensions are converted from metric values.

California Standard “Voided” Slab

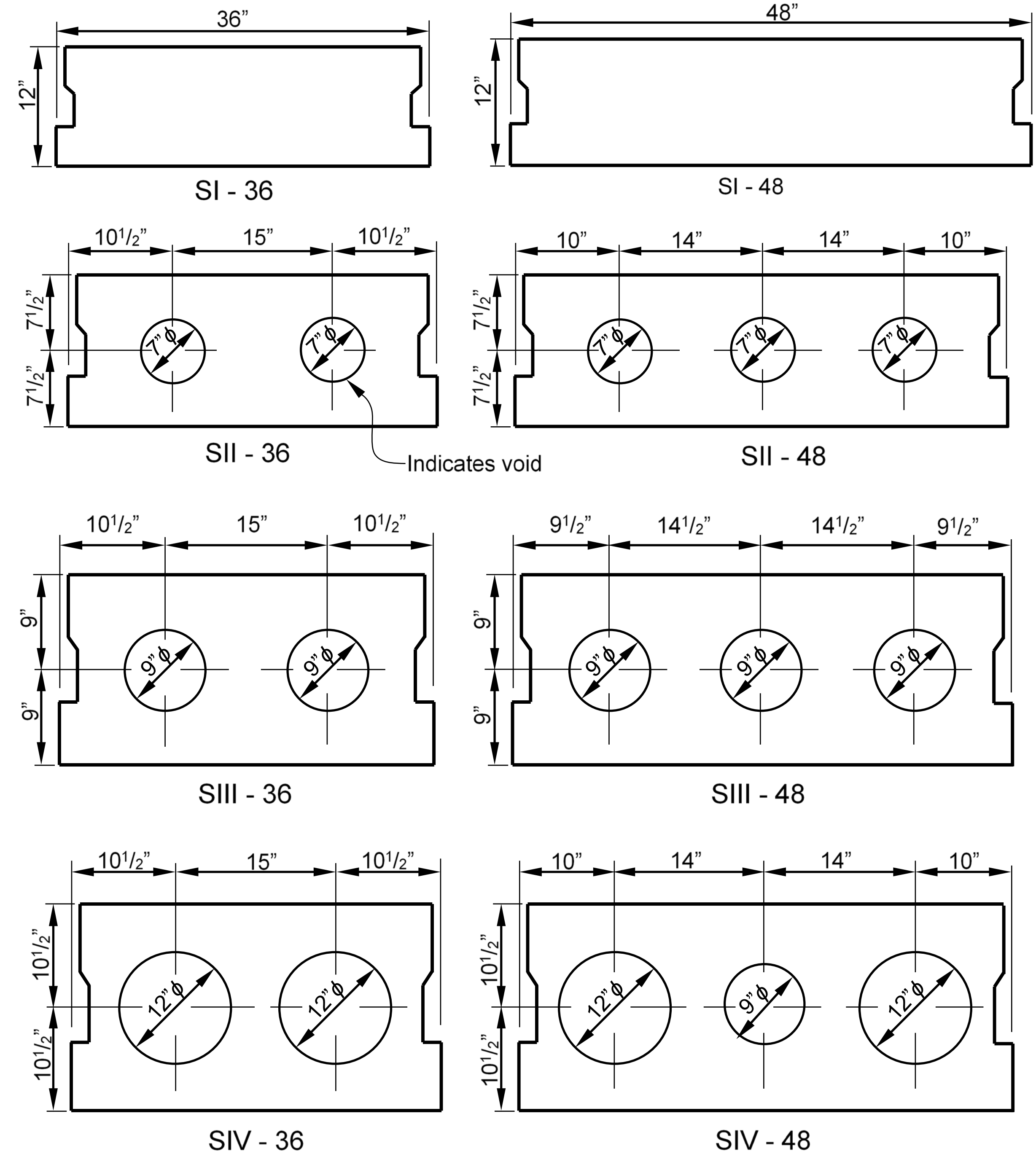
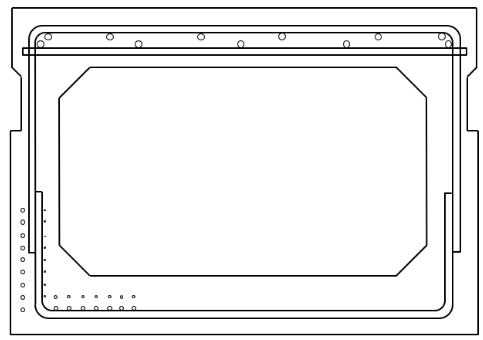
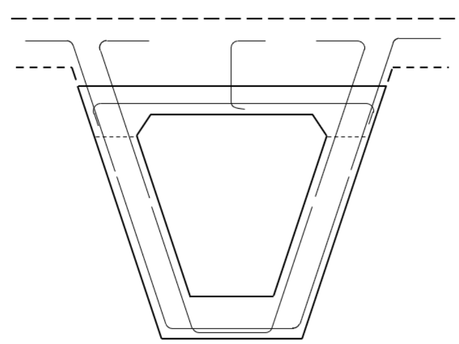


Figure 5.3.4.1-6 California Standard “Voided” Slab Typical Sections

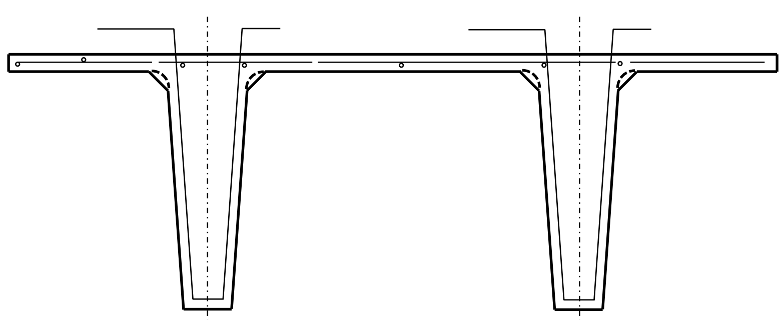
5.3.4.2 Non-Standard Precast Girder Sections



Precast Box Girder



Precast Delta Girder



Precast Double T Girder

Figure 5.3.4.2-1 Non-Standard Precast Girder Sections

5.3.4.3 Girder Types and Span Lengths

Typical precast prestressed girder types and their possible and preferred span lengths are listed in Table 5.3.4.3.1 below. The preferred span lengths take into account the girder weight for transportation and erection. Girder efficiency is generally maximized when the preferred span length is used. In addition, specific bridge types and their preferred span lengths are listed in Table 5.3.4.3.2.

Table 5.3.4.3.1 Typical Girder Shape and Span Length

Girder Type	Possible Span Length	Preferred Span Length
California I-Girder	50’ to 125’	50’ to 95’
California Bulb-Tee Girder	80’ to 150’	95’ to 150’
California Bath-Tub Girder	80’ to 150’	80’ to 120’
California Wide-Flange Girder	80’ to 200’	80’ to 180’
California Voided Slab	20’ to 70’	20’ to 50’
Precast Box Girder	40’ to 120’	40’ to 100’
Precast Delta Girder	60’ to 120’	60’ to 100’
Precast Double T Girder	30’ to 100’	30’ to 60’

Table 5.3.4.3.2 Typical Precast Bridge Type and Span Length

Bridge Type	Possible Span Length	Preferred Span Length
Precast-pretensioned Girder	30’ to 200’	30’ to 180’
Post-tensioned Spliced Precast Girder	100’ to 325’	120’ to 250’
Segmental Precast-pretensioned Girder	200’ to 450’	250’ to 400’

The precast industry should be contacted to verify that a specific girder type, depth, and length can be successfully delivered to the bridge site, especially if the site does not have freeway access or is in a remote area. A standard form that is posted on the DES BD intranet website should be used to facilitate the precast industry review.

5.3.4.4 Miscellaneous Girder Section Considerations

- For the design of Wide-Flange girders, a 3’-6” deep section is feasible from certain precast manufacturers.
- For the design of voided slab sections, it is more economical to use the same section widths as much as possible. Minor adjustments to the standard 36” and 48” section widths can be easily accommodated by the manufacturer.
- For non-spliced girders, the Wide-Flange girder section is recommended over the Bulb-Tee girder section due to its design efficiency, transportation, and erection stability.
- For spliced girders, both Wide-Flange girders and Bulb-Tee girders need to be considered. Sometimes Bulb-Tee girders are advantageous over Wide-Flange girders for their comparably reduced weight.
- When the AASHTO-CA BDS-8 Table 2.5.2.6.3-1 criteria is used for the minimum depth of adjacent box girders (beams), the deflection recommendation in this BDM that prohibits sagging may control the design for long spans. Therefore, it is recommended to use at least a 0.035 depth-to-span ratio for span lengths over 90 feet.

5.3.5 MATERIALS

The minimum concrete compressive strength at release, f'_{ci}, and the minimum 28-day concrete compressive strength, f'_c, for precast prestressed girders shall be determined during design, shown on the plans, and rounded to the nearest 0.1 ksi. The specified concrete strengths, f'_{ci} and f'_c, shall not be less than 4.0 ksi. For high strength concrete girders, f'_{ci}, may be as large as 7.0 ksi, and f'_c may be as high as 10.0 ksi. In special circumstances, a f'_{ci} up to 8.5 ksi at release can be achieved with extended curing time.

The maximum allowable temporary tensile stress at transfer in areas other than the precompressed tensile zone, such as the top flange of each girder end, is permitted to be a maximum of $0.24\lambda\sqrt{f'_{ci}}$ (ksi) as long as bonded reinforcement or prestressing strand is provided to resist the tensile force in the concrete per AASHTO-CA BDS-8 Table 5.9.2.3.1b-1. Otherwise, the allowable stress is limited $0.0948\lambda\sqrt{f'_{ci}}$ (ksi). Providing the bonded reinforcement or prestressing strand is recommended because it typically reduces the required f'_{ci}.

Precast prestressed concrete girder design is normally based on the use of 0.6-inch diameter, 270 ksi low relaxation strands. The use of 0.5-inch diameter strands is less common. 0.375-inch diameter strands are typically used for stay-in-place precast deck panels. If epoxy coated prestressing strands are required, a note indicating this should be shown on the design plans, and the corresponding section of the Standard Specifications should be used.

Deformed welded wire reinforcement (WWR) is permitted for use as shear reinforcement. WWR shall conform to ASTM A1064 per the Caltrans Standard Specifications, with a recommended maximum allowed yield strength of 60 ksi. Reinforcing bars may be used to supplement WWR.

5.3.6 FLEXURAL DESIGN AND PRESTRESSING FORCE

Flexural design for precast prestressed concrete girders includes the design of the girders at the service limit state to satisfy stress limits, followed by a check of the girders at the strength limit state to provide adequate moment resistance under ultimate conditions. In multi-span bridges, the mid-span section is usually subjected to positive moments and designed as simply supported for loads resisted in a non-composite condition, and as continuous for loads resisted in a composite condition. At bent locations, the superstructure is generally designed for continuity under live load and superimposed dead loads. Except in spliced-girder systems, negative moment reinforcement is required in the deck over the bents to resist these loads. The member at the bent locations is treated as a conventionally reinforced concrete section.

For continuous precast girder spans on bridges with a drop or integral rectangular bent caps, or for post-tensioned spliced girders joined at bent caps, bottom prestressing strands or reinforcing bars shall be extended into the end diaphragms as required per

SDC 2.0 Article 7.2.1.2. Extended bottom strands or reinforcing bars may be hooked between girder ends in the bent diaphragms to ensure adequate development. These strands or reinforcing bars are designed to resist earthquake-induced forces. Restraint moments due to creep, shrinkage, and temperature change at the girder/diaphragm connection need not be considered per AASHTO-CA BDS-8 Article 5.12.3.3.1. However, a detailed restraint moment analysis for the design of the girder/diaphragm connection should be made if a girder's age at the time continuity is established is less than 28 days.

For the service limit state design, flexural tensile and compressive stresses are limited to the values specified in AASHTO-CA BDS-8 Article 5.9.2. Prestress losses are calculated per AASHTO-CA BDS-8 Article 5.9.3. The refined estimates method for time-dependent losses is recommended for the final girder design. AASHTO-CA BDS-8 Table 3.4.1-4 shall apply. For the strength limit state design, flexural resistance is provided per AASHTO-CA BDS-8 Article 5.6.3.2. In addition, the strain compatibility approach may be used if more precise calculations are required per AASHTO-CA BDS-8 Article 5.6.3.2.5.

To satisfy stress limits, harping of strands or debonding of strands in the girders is often used in design and construction.

When harping strands, the centroid of the prestressing strands varies along the member length. In this case, the centroid of the prestressing strands is determined for design purposes and provided on the design plans. In addition, the centroid of harped strands used for design should allow a ±3-inch tolerance at girder ends to aid the manufacturer in establishing final strand positions that will accommodate the strand anchorages. This requires the designer to investigate the design for multiple scenarios. A tabulation of different P_{jack} values for individual girders should be shown when P_{jack} varies between girders by more than two strands.

Debonding a portion of the strands at the girder ends may also be used to satisfy temporary tensile stress limits at release per the AASHTO-CA BDS-8 Bridge Design Specifications Table 5.9.2.3.1b-1. Typically, strands that are straight along the entire length of the girder are selected for debonding to increase fabrication efficiency and economy. The number of these partially debonded strands is limited by AASHTO-CA BDS-8 Article 5.9.4.3.3. The prestressing strand pattern of a girder needs to be shown on the plans when partially debonded strands are specified. Per AASHTO-CA BDS-8 Table 5.9.2.3.2b-1, there is no tensile stress limit at the service limit state for areas other than the precompressed tensile zone. The top of each girder end is considered as an area other than the precompressed tensile zone. To help reduce the number of debonded strands in the bottom of the girder and to provide additional stability during transport and handling, strands in the top of the girder may be incorporated. Some of these top strands may be debonded along the middle portion of the girder to help maintain the effectiveness of the bottom strands in the precompressed tensile zones. One or two blockouts at the top of the girder are normally required to allow the top strands to be cut. The top strands should be cut before cast-in-place intermediate diaphragms or the concrete deck are placed. Top strands may be debonded at release if more initial upward girder deflection at mid-span is desired.

A girder design may require additional reinforcement to satisfy the strength limit state requirements. However, adding mild reinforcement may not be feasible or desirable due to space limitations. In such cases, the number of prestressing strands may be increased to satisfy the strength limit state while the P_{jack} force is solely used satisfy the service limit state. This design can be achieved by designing for a jacking stress less than $0.75f_{pu}$. Unstressed strands are not recommended for the purpose of increasing moment capacity.

5.3.7 SHEAR DESIGN

Shear design is performed using the sectional method specified by AASHTO-CA BDS-8 5.7.3.4 and 5.7.3.4.2. For skewed bridges, the live load shear demand is magnified in accordance with AASHTO-CA BDS-8 Article 4.6.2.2.3c. A shear correction factor is not required for dead loads. Due to the need for field bending stirrups, #4 or #5 stirrups are preferred. Stirrups larger than #6 should not be used. As shown on the bridge standard details sheets, outward bent stirrups are commonly used. Stirrups with inverted U-bars combined with hat bars may be considered with certain conditions. Interface shear is designed based on the shear friction provisions of AASHTO-CA BDS-8 Article 5.7.4. For precast girder bridges, interface shear design is considered across the interface between two concretes cast at different times, such as the top of the girder and the bottom of the deck slab, at the interface between girder ends and diaphragms at abutments or bents, and at spliced construction joints for spliced girders. A $^{1}/_{4}$-inch amplitude intentionally roughened surface or shear key at construction joints shall be provided to increase the friction factor and thus enhance the interface shear capacity.

5.3.8 END BLOCKS AND END SPLITTING RESISTANCE

Due to an increase in girder weight and overall cost, end blocks should only be used where required, such as for girders with dapped ends at inverted-tee bent cap locations and at anchorage areas for post-tensioned spliced precast girders. If an end block is required, it is recommended that only one end block be used per girder. The block length should be 1.0 to 1.5 times the height of the girder. If two end blocks are required, the design plans should specify the minimum end block lengths to provide the manufacturer flexibility to adjust forms.

The end splitting (i.e., bursting) resistance in the pretensioned anchorage zone shall be designed based on AASHTO-CA BDS-8 Article 5.9.4.4.1. For spliced precast girders where post-tensioning is directly applied to the girder end block, general zone reinforcement shall be provided in the end block per AASHTO-CA BDS-8 Article 5.9.5.6.5.

5.3.9 DEFLECTION AND CAMBER

Accurately predicting girder deflection is difficult because the concrete modulus of elasticity increases with time as the strength of the concrete increases. In addition, the effects of creep, shrinkage, and long-term prestress losses on deflection are difficult to calculate with accuracy.

For precast pretensioned girders and post-tensioned spliced girders, the mid-span design deflection due to dead load, prestressing, creep, and shrinkage should not be downward at release or after deck placement. Sagging girders are not aesthetically pleasing and may be perceived as being under-designed.

The plans should provide the instantaneous mid-span downward girder deflections due to the weights of the following elements (if present):

- Deck
- Barrier
- Sound wall
- Diaphragms
- Specified deck wearing surface, not including future wearing surface

These deflections are added to the deck contour grades when setting deck screed line grades during construction. Instantaneous deflections are used because it is assumed that no significant long-term deflection occurs after the deck hardens.

An estimate for the mid-span upward girder deflection (often referred to as “camber”) just prior to deck placement, including self-weight, prestressing, creep, and shrinkage, needs to be calculated during the design and should be shown on the plans as an estimate only. This estimate is used when determining the structure depths shown on the plans. As part of the shop drawings, the Contractor is required to independently calculate and provide this upward girder deflection (see Standard Specifications 51-4.01C(2)(b)). If the Contractor’s calculated deflection, or preferably, the anticipated deflection based on measurement of the already-fabricated girder, is different from the estimate shown on the plans, the structure depth at supports and the corresponding seat elevations may need to be revised in construction (see Standard Specifications 51-4.03B).

For typical girders (e.g., “I”, “Bulb-Tee,” and “Wide-Flange”) with a cast-in-place deck, a haunch is used to make up the elevation difference between the bottom of the deck and the top of the girder (see Figures 5.3.9.1, 5.3.9.2 and 5.3.9.3). The haunch thickness typically varies along the length of the girder, as well as along the width of the flange, and is usually maximized at the supports or at mid-span. Any haunch thickness variation along the girder results in a varying structure depth. The amount of variation depends on the following:

- Roadway profile grade along the girder centerline
- Roadway cross slope normal to the girder centerline
- Upward girder deflection just prior to deck placement
- Instantaneous downward girder deflections as listed above.

Because of the likely difference between the estimated and actual upward girder deflection just prior to deck placement, conservative design assumptions for haunch weight and structure depth should be used. It should be assumed that the actual upward girder deflection just prior to deck placement may be as little as 50% or as much as 150% of the estimated value in design, with a minimum assumed difference being 1” less and 1” more. The larger assumed upward girder deflection should be used for determining the minimum haunch thickness at mid-span so the girder does not encroach into the deck. The smaller assumed upward girder deflection should be used for determining the haunch weight and the minimum vertical clearance. The minimum haunch thickness at supports needs to be greater than or equal to 0” so the girder does not encroach into the deck at the supports.

The plans should show the following based on the designer’s calculations:

- Structure depth at supports
- Structure depth at mid-span.

Since the contractor is allowed to adjust the bearing seat elevations and structure depths during construction to accommodate the actual upward girder deflection, the structure depth at the centerline of the girder for capacity calculations should be conservatively based on a summation of the following:

- Deck thickness
- Deck cross-slope times one-half the top flange width
- Girder depth.

See Figure 5.3.9.4 for an example of how to show the structure depths on the typical section.

For adjacent box girders and slabs with a cast-in-place deck, the deck thickness will likely vary to make up for the elevation difference between the roadway profile grade and the top of the girder (see Figures 5.3.9.5 and 5.3.9.6). Most of the above recommendations for typical girders can be applied to adjacent box girders and slabs, where the additional deck thickness can be treated similarly to a haunch. For adjacent box girders and slabs without a cast-in-place deck, a non-structural overlay is required to make up the difference between the roadway profile grade and the top of the girder. Any necessary variation in overlay thickness needs to be considered in the design.

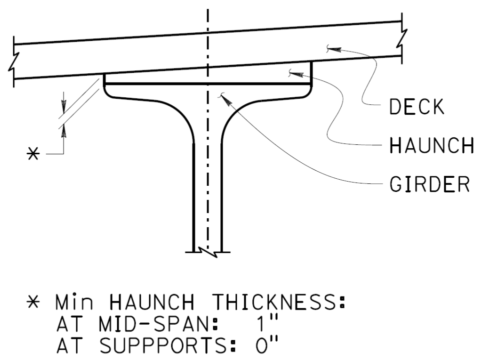


Figure 5.3.9.1 – Section Showing Haunch Thickness Variation Transversely to Centerline of Girder

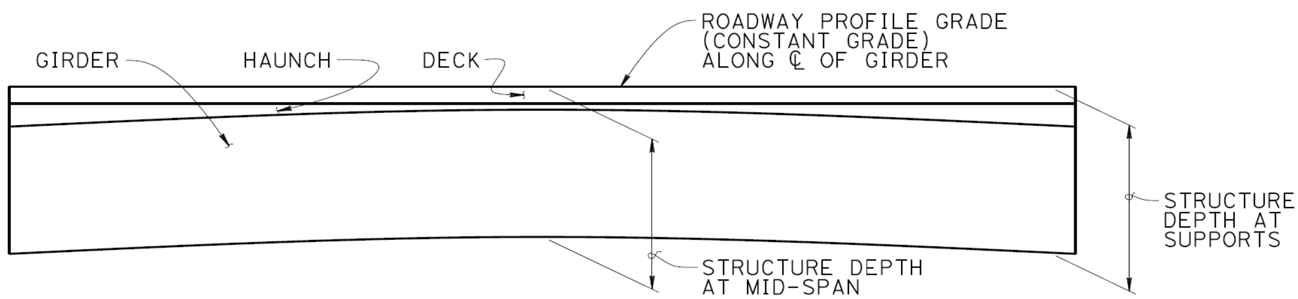


Figure 5.3.9.2 – Elevation Showing Possible Haunch and Structure Depth Variation for Roadway Profile with Constant Grade

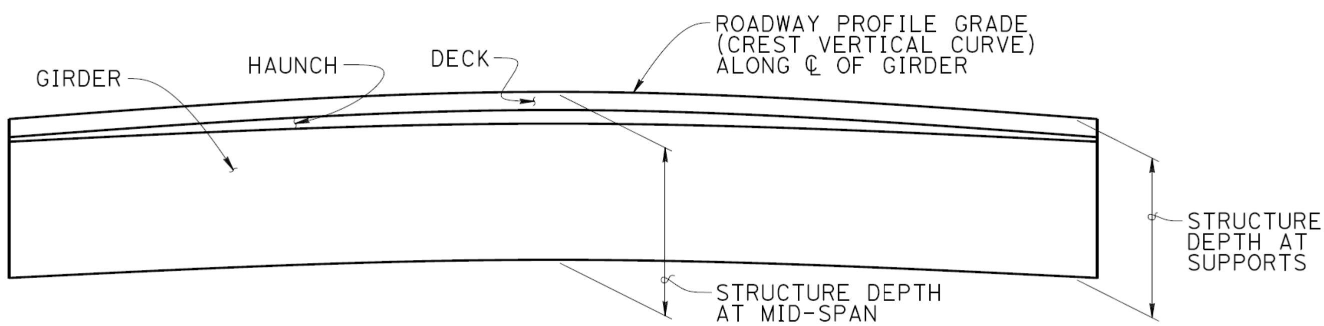


Figure 5.3.9.3 – Elevation Showing Possible Haunch and Structure Depth Variation for Roadway Profile with Crest Vertical Curve

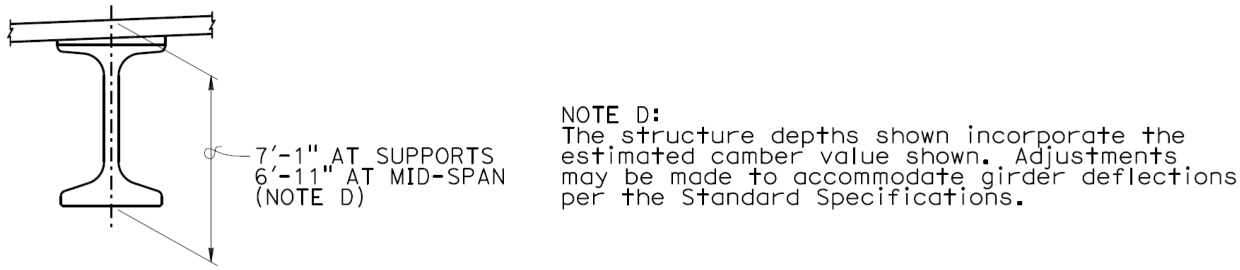


Figure 5.3.9.4 – Example Detail Showing Structure Depths on Typical Section

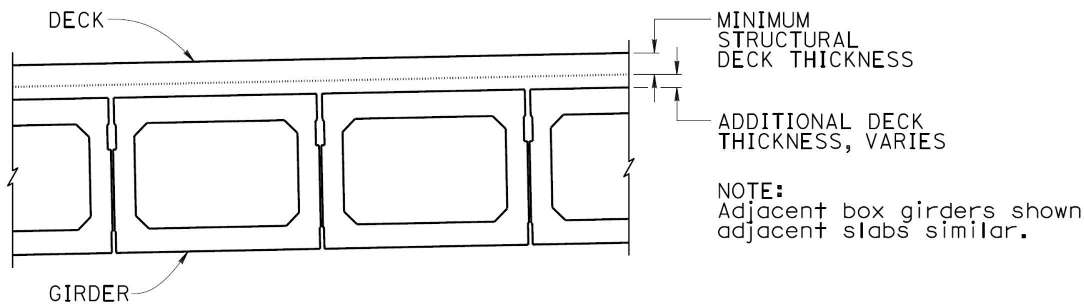


Figure 5.3.9.5 – Section Showing Deck Thickness Variation on Adjacent Box Girders and Slabs

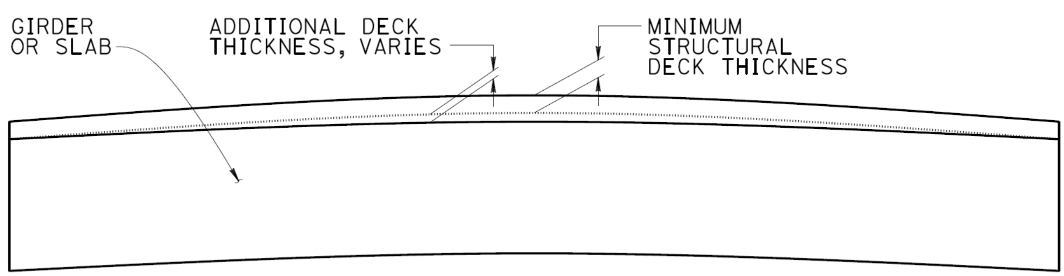


Figure 5.3.9.6 – Elevation Showing Possible Deck Thickness Variation on Adjacent Box Girders and Slabs

5.3.10 INTERMEDIATE DIAPHRAGMS

Although intermediate diaphragms may not be required per AASHTO-CA BDS-8 Article 5.12.4, they improve the distribution of loads between girders and help stabilize the girders during construction and under any potential high load hits. Girder lengths over 80 feet are recommended to use one intermediate diaphragm at mid-span. Girder lengths over 120 feet should use two or three evenly spaced intermediate diaphragms. Intermediate diaphragms should be used for all highly skewed bridges. For bridges with skews of 20 degrees or less, either normal or skewed intermediate diaphragms may be provided. For bridges with skews greater than 20 degrees, intermediate diaphragms normal to the girders are preferred as they can be staggered.

5.3.11 STAY-IN-PLACE PRECAST PRESTRESSED CONCRETE DECK PANELS

STP 9.1 addresses the applicability and design criteria of Stay-In-Place Precast Prestressed Concrete Deck Panels (PDPs) used for constructing concrete decks on precast girder superstructures. PDPs are partial-depth precast, pretensioned concrete deck panels that span between girders and are topped with cast-in-place concrete to complete the deck. Standard details sheets and User Guides are provided at the Caltrans' Bridge Standard Details website.

5.3.12 DECK AND DIAPHRAGM PLACEMENT SEQUENCE

Caltrans Standard Specifications 51-1.03D(2) "Concrete Bridge Decks and Diaphragms" describes the standard deck and diaphragm placement sequence for precast girder construction. It is recommended that the standard sequence be followed. If the intended sequence is different from that described in the Standard Specifications, it should be shown on the plans and shall be accounted for in the design.

5.3.13 SPLICED PRECAST PRESTRESSED GIRDERS

A spliced girder is a long precast prestressed concrete girder that is fabricated in segments. These segments are assembled into a single or continuous girder at or near the project site. Post-tensioning is generally used to connect the girder segments longitudinally to provide continuity. The post-tensioning can be used to create an integral superstructure-to-substructure connection to meet seismic performance requirements.

Spliced girders, which combine pretensioned concrete girders with post-tensioning techniques, may offer the following benefits:

Rapid construction using precast elements reduces congestion, traffic delays, and total project cost.

Longer span lengths reduce the number of piers and minimize environmental impact.

Fewer joints in the superstructure improve structural performance, including seismic performance, reduce long-term maintenance costs, and increase bridge service life.

The use of post-tensioning for continuity minimizes bridge superstructure depth, improving vertical clearance.

The smaller amount of required falsework minimizes construction impact and improves safety for the traveling public and construction workers.

Increased girder spacing reduces the number of girder lines and total project cost.

In addition to the requirements of AASHTO-CA BDS-8 Article 5.12.3.4, some general design and construction challenges and considerations include the following:

Spliced girder bridge design normally consists of precast pretensioned girders and post-tensioning. Therefore, prestress notes for both pre-tensioning and post-tensioning shall be shown on the design plans.

Precast girders can be either spliced in place using temporary towers or spliced on the ground near the project site and then erected. The design plans shall specify the construction method one way or another.

The designer shall take into account the construction sequence and staging. Temporary supports and locations shall be considered and designed properly as these affect the girder section, span length, and pre-tensioning and post-tensioning forces. Temporary support locations and the reactions for each stage of construction shall be shown on design plans or provided in the Special Provisions. The Contractor is responsible for the design of the temporary supports.

The stresses during temporary and final conditions as well as during the service limit state shall be addressed in design considering both temporary and final concrete stresses in girder segments at each stage of pre-tensioning and post-tensioning as well as under all applicable loads during construction. The strength limit state only needs to be considered for the final completed stage.

Post-tensioning may be applied to precast girders before and/or after placement of the deck concrete. When post-tensioning is applied to the girders both prior to and after placement of the concrete deck, construction is referred to as “two-stage post-tensioning”.

In general, one-stage post-tensioning is relatively simple in design and construction and is mostly used with bridge span lengths less than approximately 140 feet. Normally, applying the full post-tensioning force is desirable after the deck becomes part of the composite deck-girder section. However, applying the full post-tensioning force prior to deck placement allows for future deck replacement or can meet other project-specific requirements. In this one-stage approach, the post-tensioning force and girder compressive strength ($f_c^'$) are usually higher than when the post-tensioning is applied to the composite section or for two-stage post-tensioning.

When the bridge span length exceeds approximately 140 feet and/or the precast girders are in segments, two-stage post-tensioning typically results in a more efficient bridge system. The first-stage post-tensioning force is designed to control concrete stresses throughout the continuous span for loads applied before the second stage of post-tensioning. The second stage post-tensioning force is usually designed for superimposed dead and live loads. Benefits of the two-stage post-tensioning method include lower required pre-tensioning force, more efficient total post-tensioning force for the structure, lower $f_{ci}^'$ and $f_c^'$ required for the precast girder, and better deflection control.

Prestress losses due to the effects of pre-tensioning, post-tensioning, and possible staged post-tensioning shall be considered. Time-dependent software should be used to properly account for prestress losses associated with multiple stages.

Instantaneous deflections due to post-tensioning at different stages shall be shown on the contract plans. These deflection values, combined with precast pre-tensioned girder deflection values, are used to set screed line grades in the field.

The center of gravity of the post-tensioning tendon profile shall be shown on the design plans. Although a specific tendon placement pattern need not be provided in the bridge plans, the designer shall develop at least one workable tendon placement solution at all locations along the span, including at anchorages. The tendon duct size shall be based on an inside duct area at least 2.5 times the total area of prestressing strands (AASHTO-CA BDS-8 5.4.6.2). The outside duct diameter shall not be greater than one-half the girder web width similar to AASHTO-CA BDS-8 5.4.6.2 and shall be shown on the design plans. The maximum number of strands per tendon should not exceed nineteen 0.6”- diameter strands. In design, a maximum duct size should be assumed and taken into consideration for web shear capacity reduction per AASHTO-CA BDS-8 C5.7.2.1.

Cast-in-place closures between girder segments are usually used instead of match-cast joints. The width of a closure shall not be less than 24 inches and shall allow for the splicing of post-tensioning ducts and rebar. The web shear reinforcement over the spacing (A_v/s) within the closure shall be the larger of that provided in the adjacent girders. The face of the precast segments at closures shall be intentionally roughened or cast with shear keys in place.

5.3.14 REFERENCES

1. AASHTO. (2017). *AASHTO LRFD Bridge Design Specifications*, 8th Edition. American Association of State Highway and Transportation Officials. Washington, DC.
2. Caltrans. (2019). *California Amendments to AASHTO LRFD Bridge Design Specifications*, 8th Edition, California Department of Transportation, Sacramento, CA.
3. Caltrans. (2023). *Standard Specifications*, California Department of Transportation, Sacramento, CA.
4. Caltrans. (2019). *Seismic Design Criteria with Addendums*, Version 2.0, California Department of Transportation, Sacramento, CA.