Section 1 - BRIDGE SUPERSTRUCTURE

PC/PS CONCRETE DECK PANEL DETAILS (I GIRDER)

XS Sheet Numbers:

xs1-180-2 (xs1-180-2 shall be used in conjunction with xs1-180-3)

Description of Component:

PC/PS Partial-Depth Concrete Deck Panel Details for I Girder

Standard Drawing Features:

The project engineer (PE) is responsible for properly applying this standard drawing to the bridge and providing a Professional Engineer seal. The engineer should select the appropriate design from a range of PC/PS Partial-Depth Concrete Deck Panels shown in Table 1.

The two standard types of precast prestressed (PC/PS) partial-depth concrete deck panels (PDP) for bridge deck construction are rectangular panels and skewed panels. Rectangular panels are usually used for bridges with no skew and for inner panels of skewed bridges. Skewed panels are typically used to accommodate skews at abutments or bents. Rectangular or skewed end panel details are shown in xs1-180-3.

Panel Design

1. Table 1 - PC/PS Partial-Depth Deck Panel Design provides a standardized range of total deck thickness and strand spacing using 3/8-inch diameter Grade 270 prestressing. It also provides a transverse negative moment deck reinforcement of #5’s at 6 inches o.c., which is conservatively used for all cases to satisfy AASHTO-CA BDS 08 and Structure Technical Policies (STPs) 9.1 & 9.4 design requirements.
2. Based on the girder spacing layout of the bridge, the designer must determine specific values for the Deck Panel Table on the XS sheet, which applies to the bridge. The deck panel length shall be calculated based on the girder spacing by using Part Typical Section and deck panel details. The required PDP thickness, total deck thickness selected from the range provided in Table 1, and strand spacing need to be specified in the Deck Panel Table on the XS sheet.
3. The designer must list the total deck thickness in the Deck Panel Table on the XS Sheet.
4. The design values in Table 1 are based on the following assumptions: precast concrete properties (f'_{ci} = 4.5 ksi, f'_c = 6 ksi), prestress strand steel properties for the 3/8-inch diameter strand (Grade 270, 7-wire, low relaxation), and

mild reinforcement steel (Grade 60). The designer may increase the assumed CIP concrete compressive strength (f'_c = 4.0 ksi) and show the required strength on the “Concrete Strength and Type Limits” detail.

5. The design is also based on stressing the 3/8-inch diameter strands to 70% (not 75%) of the specified minimum ultimate tensile strength (i.e., $0.70f_{pu}$) to prevent panel cracking during de-tensioning. This corresponds to a jacking force of 16 kips per strand.
6. The designer is responsible for determining if the Project’s PDP system is within the Table 1 parameters of this User Guide and the assumptions listed above. For cases beyond those provided in this User Guide, the designer must develop a suitable design based on all the requirements.

Deck Panel Table

1. Designers must fill in the required information from this table on the XS Sheet (Girder Spacing, Panel Depth, Panel Length, Strand Spacing, and Total Deck Thickness).

Table 1 - PC/PS Partial-Depth Deck Panel Design: I Girder

Girder Spacing	PDP Thickness (in.)	Minimum Total Deck Thickness (in.)	Maximum Total Deck Thickness (in.)	Maximum Strand Spacing (in.)	Negative Moment Deck #5 Rebar Spacing (in.)
5'-0" to 8'-9"	3.75	8.0	10.25	6.0	6.0

Plan-Rectangular Panel

1. Panel length “*L*” is limited to 4′-0′′ min. and 8′-0′′ max.
2. Panel width “*W*” is limited to 4′-0′′ min. and 12′-0′′ max. The shop drawings will specify the "*W*" dimension for manufacturing and transportation efficiency.
3. Panel size (*L* and/or *W*) not in the specified range above requires a special design.
4. All panel dimensions and strand spacing are to be provided on the contractor’s shop drawings. The designer must review the shop drawings to ensure compliance with the original design and parameters of this user guide.

Section A-A

1. Panel depth “*D*” shall be 3.75 inches.
2. The top face of the panel (interface between precast and cast-in-place deck concrete) shall be textured per the Standard Specifications 51-4.02D(7).
3. PS strands are placed in the center of the panel to avoid camber due to eccentricity.

Part Typical Section

1. Total deck thickness “*T*” must be as shown in DECK PANEL TABLE. The corresponding CIP topping thickness is the difference between the total deck

and PDP thickness. Provide a minimum of 1-inch clearance between the CIP deck reinforcement and the PDP.

2. The height-to-width ratio of polystyrene camber strips shall be in accordance with the Standard Specifications 51-4.02D(7). The edge of the camber strips shall be located ¾″ min. and 1″ max from the edge of the girders.
3. The concrete deck pour is divided into Stage 1 (preliminary deck pour) and Stage 2 (remaining CIP topping pour) per Standard Specifications 51-4.03G.

Design/General Notes:

The design and details are based on AASHTO LRFD Bridge Design Specifications, 8th Edition, with CA Amendments. Section 51-4.02D(7) and Section 51-4.03G "Deck Panels" of Caltrans Standard Specifications provide information on PC/PS deck panel fabrication and construction. Designers must read these sections of the Standard Specifications.

Additional Drawings Needed to Complete PS&E:

This sheet works together with XS Sheet No. xs1-180-3 and a bridge project "TYPICAL SECTION" sheet.

Contract Specifications:

Standard Specifications 2023

Restrictions on Use of Standard Drawings:

The PE is responsible for applying the pre-designed Table 1 of this User Guide to the bridge.

Special Considerations:

Under some circumstances, PDPs are not permitted to be used. See STP 9.1. The PE must determine if the use of xs1-180-2 and xs1-180-3 is appropriate for the bridge. The PE shall stamp the xs1-180-2 and xs1-180-3 sheets with a valid California Professional Engineer License Stamp.

Design Calculations:

Selected design verification calculations are provided in the attached Appendix.

APPENDIX

DESIGN TABLE VERIFICATION FOR PARTIAL-DEPTH PRECAST/PRESTRESSED CONCRETE DECK PANELS (PDP) WITH CIP TOPPING

I Girder, Girder spacing, S = 8.75 ft; 8 in. Deck (PDP = 3.75 in., CIP = 4.25 in.); Strand spacing, s = 6 in.

1.0 OVERVIEW

1.1 Introduction

This calculation is a verification calculation (not design calculation) that demonstrates a Stay-In-Place Precast Prestressed Partial-Depth Concrete Deck Panels (PDPs) with a cast-in-place (CIP) concrete topping that satisfies the requirements of the AASHTO LRFD Bridge Design Specifications (8th Edition, 2017 with updates) for the specific parameters assumed in the Design Table shown in the PDP User Guide for XS Sheets.

The representative bridge used for this calculation is modeled after the 3-lane bridge of Example 9.10 in the PCI Bridge Design Manual (3rd Edition). The total bridge deck width is dependent on the girder spacing, and the deck slab for this verification is supported by four **California I Girders**. Caltrans Type 842 barriers with deck overhangs are included. See Figures 1.2.1-1 and 1.2.1-2 for bridge, girder, and PDP sections and plan.

The calculation uses the strip method for flexural analysis. Concrete stresses due to external loads and prestress effects are calculated using transformed sections. Time-dependent prestress losses are calculated using the LRFD Approximate Estimate Method. The scope of the calculation is shown in the Table of Contents and does not address shear per LRFD Article C4.6.2.1.6.

1.2 PDP System

1.2.1 Deck Design Parameters

The key deck design parameters for this PDP design verification are stated below, based in part on STP 9.1, which states: "PDPs shall have a thickness of at least 3.75 inches, a maximum length of 9'-0" [spanning in a transverse direction of the bridge]", and a width from 4-ft to 12-ft (in a longitudinal direction of the bridge). The deck must have an overall thickness of at least 8 inches. PDPs shall use a 3/8-inch diameter Grade 270 low-relaxation prestressing steel strand. The maximum tensile stress in the prestressing steel at release (Editorial note: i.e., immediately prior to transfer) shall not exceed 70% of the specified minimum ultimate tensile strength of the prestressing steel. Strands must be placed at the centroid of the PDP cross section so that the prestressing does not produce any eccentricity."

Girder Type/Spacing:

- Type = California I Girder
- Top flange width = 19 in.
- Girders spacing, S = 8.75 ft
- Overhang = min (0.5S, 6 ft) = 4.375 ft

Deck thickness:

- Total thickness, h_{tot} = 8 in.
- PDP thickness, h = 3.75 in.
- CIP topping thickness, t_s = 4.25 in.

PDP Length:

- Length= 8 ft
- Design Span, l = 8 ft (conservatively taken as total length)

PDP Width:

- Width = 8 ft
- Design Width, b = 1.0 ft (for calculation simplicity) Strands:
- Grade 270 low relaxation; 3/8-inch diameter
- Strands spacing, s = 6 in.; Edge Distance = 3 in.
- Number of Strands, N = 16 for 8 ft width
- Zero Eccentricity for Noncomposite PDP section.

PDP Specified Concrete Compressive Strength:

- f'_{ci} = 4.5 ksi
- f'_c = 6.0 ksi
- CIP Topping: f'_{cs} = 4.0 ksi

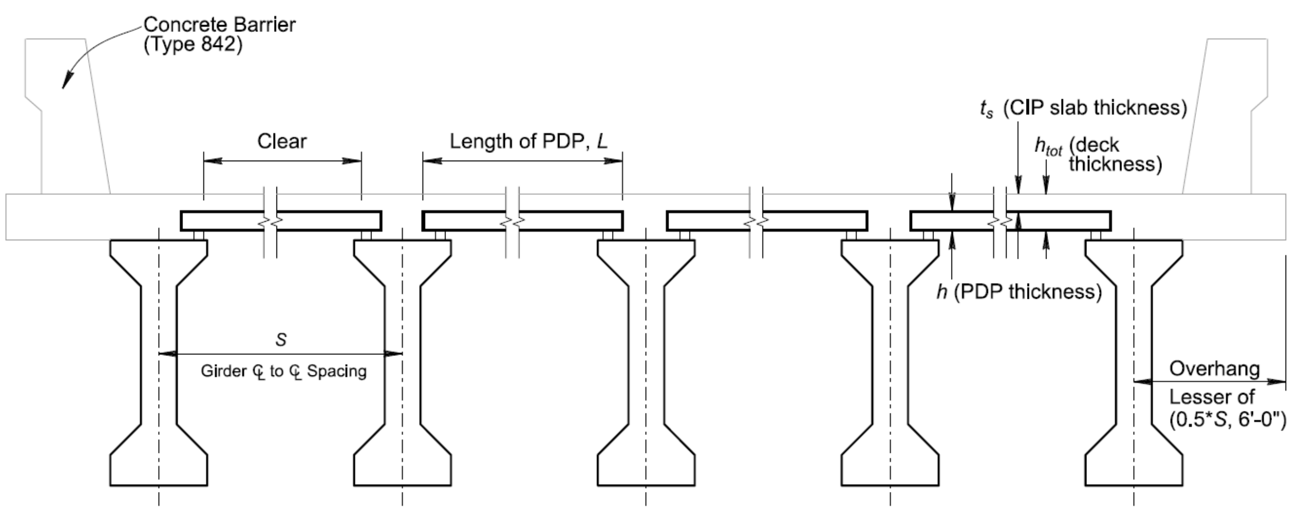
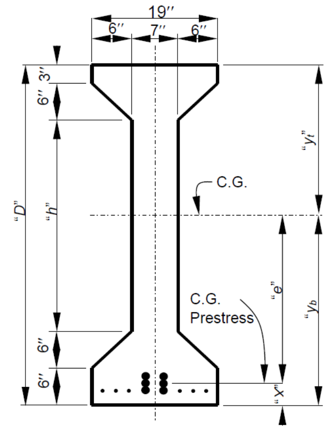
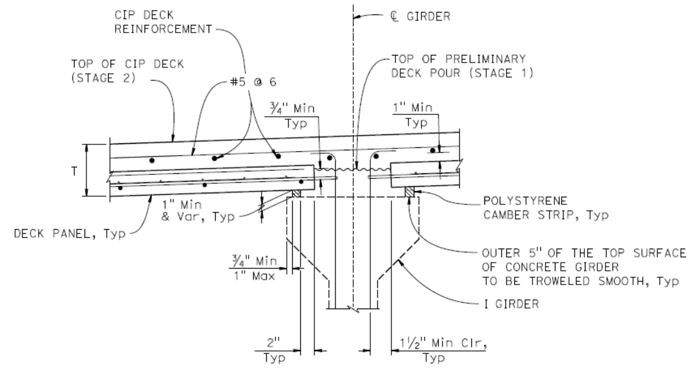


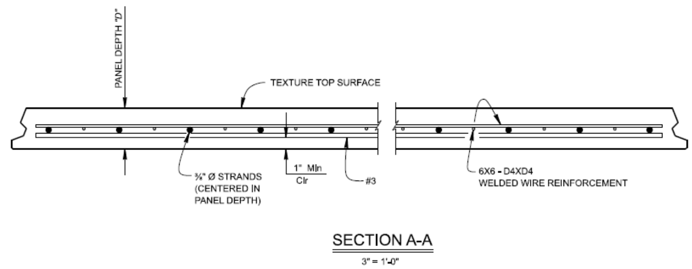
Figure 1.2.1-1. Bridge Typical Section with PDPs



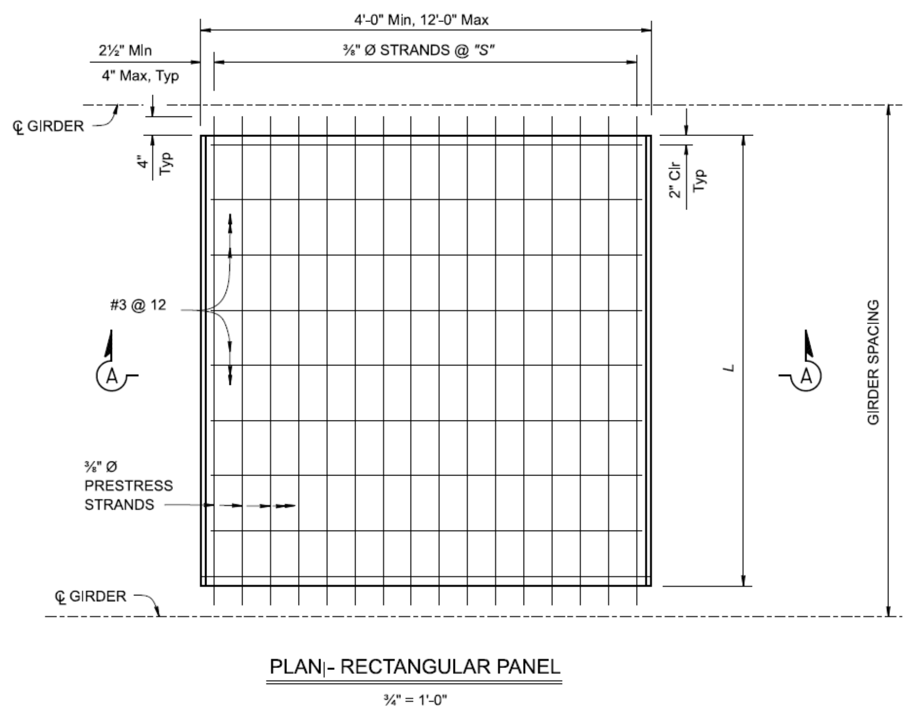
(a) I Girder Section



(b) PDP Partial Typical Section Along Span



(c) PDP Typical Section



(d) PDP Plan View

Figure 1.2.1-2 PDP Typical Details

Legend

D = Demand

C = Capacity

D/C is Demand/Capacity Ratio for Stress or Strength Checks

OK = Design check satisfied

NG = Design check not satisfied

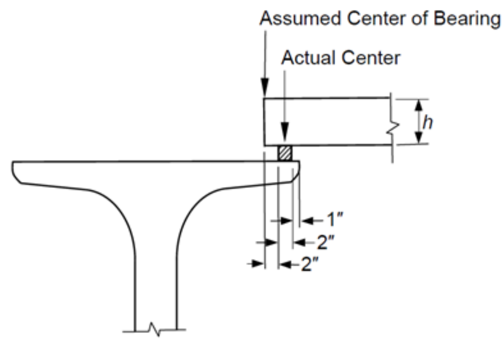


Figure 1.2.1-3 Assumed Bearing on Girder (Conservatively taking as 5 in. from the edge)

PDP Dimensions

Girder spacing, S = 8.75 ft

PDP design span length, $l = S$ - top flange width + (5 in. x 2)

$$l = S - 19 + 10 = 96 \text{ in.}$$

PDP width, w = 1 ft = 12 in. Use 1 ft unit width for calculations (8 ft total)

PDP depth, h = 3.75 in.

CIP topping slab depth, t_s = 4.25 in.

Total deck thickness, $h_{tot} = h + t_s$ = 8 in.

1.2.2 Deck Design Checks

Total deck thickness, h_{tot}

Minimum required total deck thickness = 8.0 in. [STP 9.4]

$h_{tot} \geq 8.0$ in. OK

Maximum and Minimum thickness of PDP

PDP thickness ≤ 55% of total deck depth and not less than 3.75 in.

[LRFD Art. 9.7.4.3.1; STP 9.1]

$0.55\ h_{tot}$ = 4.4 in.

h = 3.75 in. $\leq 0.55\ h_{tot}$ and $h \geq 3.75$ in. OK

1.3 Terminology

The following terminology is used in this calculation:

- **Noncomposite section** - actual PDP section
- **Noncomposite nontransformed section** - PDP section without the strands transformed (i.e., gross section)
- **Noncomposite transformed section** - PDP section with strands transformed to equivalent PDP concrete
- **Composite section** - actual PDP section with CIP topping slab and haunch
- **Composite nontransformed section** - PDP section, with CIP topping slab and haunch transformed to equivalent PDP concrete (but without strands transformed)
- **Composite transformed section** - PDP section, with CIP topping slab and haunch and strands transformed to equivalent PDP concrete

NOTE

The term "**composite**" with nontransformed or transformed section implicitly includes the transformation of CIP topping slab and haunch to equivalent PDP concrete

The term "**transformed**" refers to the transformation of the PDP strands to equivalent PDP concrete

2.0 MATERIALS

The PDP initial (specified) concrete compressive strength, f'_{ci}, of 4.5 ksi, and final specified compressive strength, f'_c, of 6.0 ksi are based on typical economical values achieved in practice. The compressive strength for the CIP topping of 4.0 ksi is also a typical value used in practice. Other material properties used for concrete (weight, E_c, etc.) and PS steel and rebar are typical industry standard values.

2.1 Cast-in-place (CIP) Concrete Topping

CIP topping slab thickness, t_s = 4.25 in. = 0.354 ft

Specified concrete compressive strength for topping slab, f'_{cs} = 4.0 ksi

CIP concrete unit weight, w_c = 0.150 kip/ft^3

2.2 Precast Deck Panels

Specified concrete compressive strength at transfer, f'_{ci} = 4.5 ksi

Specified concrete compressive strength at service, f'_c = 6.0 ksi

PDP unit weight, w_c (assumed to be same as CIP) = 0.150 kip/ft^3 [LRFD Art. 5.4.2.8]

For normal-weight concrete, the concrete density modification factor, λ, is 1.0 in LRFD Equations.

2.3 Prestressing Strands

The prestressing (PS) strands use industry standard properties, including Grade 270, low-relaxation steel. Smaller 3/8-inch diameter strands are suitably used for PDPs. The total number of strands for the assumed PDP width (8 ft) and strand edge spacing is determined. Based on this, the area of strands <u>per UNIT foot width</u> is determined and used throughout the calculations.

Nominal strand diameter for 3/8-inch strand, $d_b = \frac{3}{8}$in.

Area of one strand, A_{st} = 0.085 in.^{2}

Specified tensile strength, f_{pu} = 270 ksi

Yield strength, f_{py} = 0.9 f_{pu} = 243 ksi [LRFD Table 5.4.4.1-1]

Jacking stress, f_{pj} = 0.7 f_{pu} = 189 ksi [STP 9.1]

NOTE: This limit is intended to reduce splitting stresses at ends of PDP and thus the likelihood of related cracking.

Stress limits for PS strands

Prior to seating (short-term), f_{pj_short} = 0.9 f_{py} = 218.7 ksi [LRFD Table 5.9.2.2-1]

Immediately prior to transfer, f_{pj_max} = 0.75 f_{pu} = 202.5 ksi [LRFD Table 5.9.2.2-1]

Jacking stress, f_{pj} shall be ≤ f_{pj_max} = 189 ksi ≤ 202.5 ksi OK

[LRFD Table 5.9.2.2-1; STP 9.1]

At service limit state (after all losses), f_{pe_max} = 0.80 f_{py} = 194.4 ksi

[LRFD Table 5.9.2.2-1]

Modulus of Elasticity, E_p = 28500 ksi [LRFD Art. 5.4.4.2]

Calculate the total # of strands, *N*, based on assumed

Strand spacing, *s* = 6 in.

PDP width, w_8 = 8 ft = 96 in.

Edge distance, d_{edge} = 3 in.

$$N = \frac{(w_8 - 2 \cdot d_{edge})}{s} + 1 = 16$$

<u>Spacing limits for PS strands,</u> CA_{max} = 0.75 in. Conservative assumption for max coarse aggregate

Minimum strand spacing, s_{min} = max (1.5 in. or d_b + 1.33 CA_{max}) = 1.5 in.

[LRFD Art. 5.9.4.1, Table 5.9.4.1-1]

Maximum strand spacing, s_{max} = min (1.5 $h_{tot.}$ or 18 in.) = 12 in. [LRFD Art. 5.9.4.2]

$s_{min} \leq s$ = 6 in. $\leq s_{max}$ OK

Calculate area of PS strand per ft of PDP width, A_{ps}

$$A_{ps} = A_{st} \bullet (\frac{N}{w_8}) \bullet 12 \text{ in.} \Rightarrow \mathrm{A}_{ps} = 0.17 \text{ in.}^2$$ Per unit foot of PDP width

2.4 Reinforcing Bars

Yield strength for Grade 60 rebar, f_y = 60 ksi

Modulus of Elasticity, E_s = 29000 ksi [LRFD Art. 5.4.3.2]

3.0 LOADS, LOAD EFFECTS, AND LOAD COMBINATIONS

3.1 Dead Loads and Moments

The PDPs support their own weight, construction loads, and the weight of the CIP topping slab "noncompositely". PDPs and the CIP topping slab support superimposed dead and live loads "compositely". <u>A unit foot width of PDP and CIP topping are used.</u> Weights are distributed load along PDP in k/ft, and demands are determined at *L*/2. Where positive moment demand for the wearing surface is slightly higher away from *L*/2 due to continuity, this larger demand is conservatively combined with *L*/2 demand in design checks. This is a very minor effect. Moments at midspan of the PDP due to barrier rails and wearing surface loads are based on a continuous beam analysis, with the overhang taken as the lesser of (0.5*S*, 6 ft).

Weight of PDP: $W_{PDP} = h\, w_c$ (1 ft) = 0.047 kip/ft

Moment of PDP at *L*/2: $$M_{PDP} = \frac{W_{PDP} \bullet l^2}{8} = 0.375 \text{ kip-ft}$$

Weight of CIP topping slab: $W_{CIP} = t_s\, w_c$ (1 ft) = 0.053 kip/ft

Moment of CIP at *L*/2: $$M_{CIP} = \frac{W_{CIP} \bullet l^2}{8} = 0.425 \text{ kip-ft}$$

3.2 Barrier and Wearing Surface Loads and Moments

Moments at midspan of the PDP due to barrier rails and wearing surface loads are based on a continuous beam analysis, with the overhang taken as the lesser of (0.5S, 6 ft).

Barrier

Overhang = min (0.5S, 6 ft) = 4.375 ft

$W_{Barrier}$ = 0.656 kip/ft

M_b = 0.574 kip-ft

Moment at L/2 due to point load of barrier rail

(imported from structure analysis)

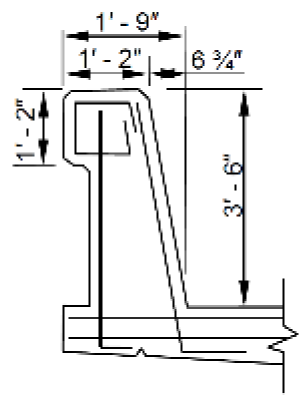


Figure 3-1 Caltrans Barrier Type 842

Wearing Surface

Future wearing surface (3 in. additional concrete), t_{ws} = 3 in.

Weight of wearing surface, $W_{ws} = t_{ws}\, w_c$ (1ft) = 0.0375 kip/ft

Moment at L/2 due to wearing surface uniform load, M_{ws} = 0.143 kip-ft (imported from structural analysis)

3.3 Construction Load and Moment

Construction load, p_{cl}, of 50 psf (applied to PDP and CIP topping) [LRFD Art. 9.7.4.1]

p_{cl} = 0.050 ksf

Weight of construction load, $w_{cl} = p_{cl}$ (1ft) = 0.05 kip/ft

Moment of construction load, $M_{const} = w_{cl}\dfrac{l^2}{8} = 0.4$ kip-ft

NOTE:

LRFD Article 9.7.4.1 requires:

1) Unfactored stress check with PDP concrete stresses limited to $0.65f'_c$ in compression or modulus of rupture in tension, and steel stress limited to $0.75f_y$, and
2) Elastic deflection due to PDP self-weight and CIP topping not to exceed L/180 nor 0.5 in.

In addition, LRFD Article 3.4.2.1 requires construction loads to be added in the Strength Load Combination I with a load factor not less than 1.5. Wet concrete deck and PDP should be considered DC loads.

3.4 Live Load and Moment

LRFD Article 3.6.1.3.3 states that for decks where the primary strips are transverse, and their span does not exceed 15 ft, the transverse strips are designed for the axle loads of the design truck (32.0-kip axle) or the design tandem.

Multiple Presence Factor

Single Truck:	SF_{st} = 1.2	[LRFD Table 3.6.1.1.2-1]
Two Trucks:	SF_{tt} = 1.0	[LRFD Table 3.6.1.1.2-1]
Dynamic Load Allowance:	DLA = 33%	[LRFD Table 3.6.2.1-1]

LRFD Table A4-1 (Deck Slab Design Maximum Live Load Moments) gives the values of maximum positive (and negative) bending moments for different deck slab spans (Girder spacing, *S*). This table is valid for decks supported on at least three girders and having a bridge deck width measured between the centerlines of the exterior girders of not less than 14 ft. Multiple presence factors and the dynamic load allowance are <u>included</u> in the tabulated values. (Values of negative bending moments provided by this table do not apply to the deck overhang.)

For the deck under consideration, the maximum positive bending moment, with dynamic allowance, is taken from AASHTO Table A4-1 (next page) as follows:

For S = 8.75 ft => M_{LL} = 6.14 kip-ft

Positive Live Load Moment per Unit Width for PDP Design

Table A4-1—Maximum Live Load Moments per Unit Width, kip-ft/ft

S	Positive Moment	Negative Moment						
		Distance from CL of Girder to Design Section for Negative Moment						
		0.0 in.	3 in.	6 in.	9 in.	12 in.	18 in.	24 in.
4 ft –0 in.	4.68	2.68	2.07	1.74	1.60	1.50	1.34	1.25
4 ft –3 in.	4.66	2.73	2.25	1.95	1.74	1.57	1.33	1.20
4 ft –6 in.	4.63	3.00	2.58	2.19	1.90	1.65	1.32	1.18
4 ft –9 in.	4.64	3.38	2.90	2.43	2.07	1.74	1.29	1.20
5 ft –0 in.	4.65	3.74	3.20	2.66	2.24	1.83	1.26	1.12
5 ft –3 in.	4.67	4.06	3.47	2.89	2.41	1.95	1.28	0.98
5 ft –6 in.	4.71	4.36	3.73	3.11	2.58	2.07	1.30	0.99
5 ft –9 in.	4.77	4.63	3.97	3.31	2.73	2.19	1.32	1.02
6 ft –0 in.	4.83	4.88	4.19	3.50	2.88	2.31	1.39	1.07
6 ft –3 in.	4.91	5.10	4.39	3.68	3.02	2.42	1.45	1.13
6 ft –6 in.	5.00	5.31	4.57	3.84	3.15	2.53	1.50	1.20
6 ft –9 in.	5.10	5.50	4.74	3.99	3.27	2.64	1.58	1.28
7 ft –0 in.	5.21	5.98	5.17	4.36	3.56	2.84	1.63	1.37
7 ft –3 in.	5.32	6.13	5.31	4.49	3.68	2.96	1.65	1.51
7 ft –6 in.	5.44	6.26	5.43	4.61	3.78	3.15	1.88	1.72
7 ft –9 in.	5.56	6.38	5.54	4.71	3.88	3.30	2.21	1.94
8 ft –0 in.	5.69	6.48	5.65	4.81	3.98	3.43	2.49	2.16
8 ft –3 in.	5.83	6.58	5.74	4.90	4.06	3.53	2.74	2.37
8 ft –6 in.	5.99	6.66	5.82	4.98	4.14	3.61	2.96	2.58
8 ft –9 in.	6.14	6.74	5.90	5.06	4.22	3.67	3.15	2.79
9 ft –0 in.	6.29	6.81	5.97	5.13	4.28	3.71	3.31	3.00
9 ft –3 in.	6.44	6.87	6.03	5.19	4.40	3.82	3.47	3.20
9 ft –6 in.	6.59	7.15	6.31	5.46	4.66	4.04	3.68	3.39
9 ft –9 in.	6.74	7.51	6.65	5.80	4.94	4.21	3.89	3.58
10 ft –0 in.	6.89	7.85	6.99	6.13	5.26	4.41	4.09	3.77
10 ft –3 in.	7.03	8.19	7.32	6.45	5.58	4.71	4.29	3.96
10 ft –6 in.	7.17	8.52	7.64	6.77	5.89	5.02	4.48	4.15
10 ft –9 in.	7.32	8.83	7.95	7.08	6.20	5.32	4.68	4.34
11 ft –0 in.	7.46	9.14	8.26	7.38	6.50	5.62	4.86	4.52
11 ft –3 in.	7.60	9.44	8.55	7.67	6.79	5.91	5.04	4.70
11 ft –6 in.	7.74	9.72	8.84	7.96	7.07	6.19	5.22	4.87
11 ft –9 in.	7.88	10.01	9.12	8.24	7.36	6.47	5.40	5.05
12 ft –0 in.	8.01	10.28	9.40	8.51	7.63	6.74	5.56	5.21
12 ft –3 in.	8.15	10.55	9.67	8.78	7.90	7.02	5.75	5.38
12 ft –6 in.	8.28	10.81	9.93	9.04	8.16	7.28	5.97	5.54
12 ft –9 in.	8.41	11.06	10.18	9.30	8.42	7.54	6.18	5.70
13 ft –0 in.	8.54	11.31	10.43	9.55	8.67	7.79	6.38	5.86
13 ft –3 in.	8.66	11.55	10.67	9.80	8.92	8.04	6.59	6.01
13 ft –6 in.	8.78	11.79	10.91	10.03	9.16	8.28	6.79	6.16
13 ft –9 in.	8.90	12.02	11.14	10.27	9.40	8.52	6.99	6.30
14 ft –0 in.	9.02	12.24	11.37	10.50	9.63	8.76	7.18	6.45

Table 3.4-1. Live Load Moment per Unit Width
(AASHTO LRFD Table A4-1)

3.5 Load Factors and Load Combinations

Total factored load is taken as:

$$Q = \sum n_i \gamma_i Q_i$$ [LRFD Eq. 3.4.1-1]

n_i = a load modifier relating to ductility, redundancy, and operational importance, taken as 1.0 here for typical bridges [LRFD Art. 1.3.2]

γ_i = load factor [LRFD Eq. 3.4.1-1]

Q_i = force effects from specified loads

Investigating different limit states given in LRFD Article 3.4.1, the following limit states are applicable:

<u>Service I: Check compressive stress in prestressed concrete components</u>

Q = 1.00(DC+DW) + 1.00(LL+IM) [LRFD Tables 3.4.1-1]

This load combination is the general combination for services limit state stress checks and applies to all conditions other than Service III.

<u>Service III: Check tensile stresses in prestressed concrete components</u>

Q = 1.00(DC+DW) + 1.00(LL+IM) [LRFD Tables 3.4.1-1 and 3.4.1-4]

This load combination for Service III stress checks applies only to tension in prestressed concrete structures to control cracks.

NOTE: Per LRFD Table 3.4.1-4, a load factor of 1.00 (not 0.80) is applied to (LL+IM) for Service III because the approximate loss (not refined loss) approach (with transformed section), is used for long-term losses.

<u>Strength I: Check ultimate strength</u> [LRFD Tables 3.4.1-1 and 3.4.1-2]

Maximum Q = 1.25(DC) + 1.50(DW) + 1.75(LL+IM)

Minimum Q = 0.90(DC) + 0.65(DW) + 1.75(LL+IM)

This load combination is the general load combination for strength limit state design. An additional strength check is required for construction with PDPs per LRFD 3.4.2.1, as shown later.

NOTE: For simple-span bridges, the maximum load factors produce maximum effects. Minimum load factors for dead load (DC) and wearing surface (DW) are used when dead load and wearing surface stresses are opposite to those of the live load.

<u>Fatigue</u> [LRFD Art. 9.5.3 and 5.5.3.1]

Fatigue need not be investigated for concrete deck slabs in multi-beam bridges.

4.0 SECTION PROPERTIES FOR PANEL SYSTEM

4.1 Noncomposite Nontransformed (Gross) PDP Section

NOTE: All calculations are for a **unit foot of PDP width**, i.e., b = 12 in.

b = 12 in. h = 3.75 in.

A_g = gross area of PDP cross section $A_g = b * h$ = 45 in.^{2}

I_g = moment of inertia (about the centroid) of the PDP

$$I_g = \frac{b \bullet h^3}{12} = 52.73 \text{ in.}^4$$

S_b = section modulus for the extreme bottom fiber of PDP

$$S_b = \frac{b \bullet h^2}{6} = 28.13 \text{ in.}^3$$

S_t = section modulus for the extreme top fiber of PDP

$$S_t = \frac{b \bullet h^2}{6} = 28.13 \text{ in.}^3$$

E_c = modulus of elasticity, ksi = 120000 K_1 $(w_c)^{2.0}$ $(f'_{ci})^{0.33}$ [LRFD Eq. 5.4.2.4-1]

Where:

K_1 = correction factor for source of aggregate taken as 1.0 K_1 = 1

w_c = unit weight of concrete

For 5 ksi < f'_c < 15 ksi, $w_c = 0.140 + 0.001f'_c$ [LRFD Table 3.5.1-1]

However, for precast concrete, simply use w_c =0.150 kcf, representative of PDPs:

E_{ci} and E_c for PDP

$$E_{ci} = 120000(1.0)(0.150)^{2.0}(4.5)^{0.33} = 4435 \text{ ksi}$$

$$E_c = 120000(1.0)(0.150)^{2.0}(6.0)^{0.33} = 4877 \text{ ksi}$$

4.2 Composite Nontransformed Section

The composite nontransformed section is the section for which the CIP topping slab is transformed to PDP concrete, but strands are not transformed.

<u>CIP slab</u>

$$E_{cs} = 120000(1.0)(0.150)^{2.0}(4.0)^{0.33} = 4266 \text{ ksi}$$

Modular Ratio, *n* (Converting CIP to PDP Concrete)

E_c = 4877 ksi

Modular Ratio to convert CIP slab to PDP concrete, $n_{cip} = \dfrac{E_{cs}}{E_c} = 0.8748$

Composite Nontransformed Section Properties for CIP Slab

Transformed width of CIP Slab, $b_t = n_{cip}\, b$ = 10.497 in. (12 inches CIP to narrower PDP concrete)

Transformed area of CIP Slab, $A_t = b_t\, t_s$ = 44.61 in.^{2}

Transformed moment of inertia of CIP Slab, $I_t = \dfrac{b_t \bullet t_s^{\,3}}{12} = 67.15$ in.^{4}

Figure 4-1 shows the dimensions of the composite nontransformed section.

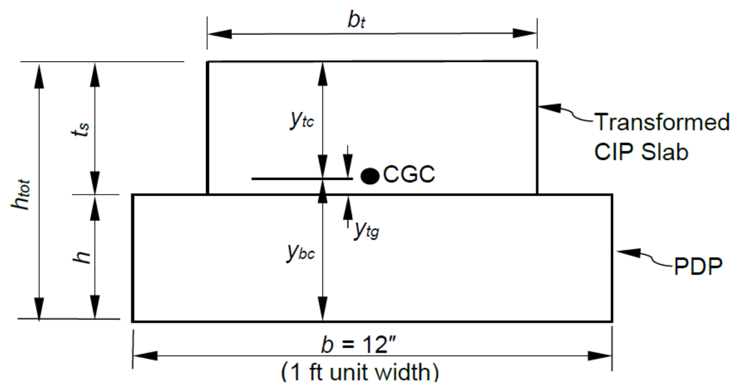


Figure 4-1. Composite Nontransformed Section

A_c = total area of composite nontransformed section

$A_c = (b\, h) + b_t\, t_s$ = 89.61 in.^{2}

y_{bc} = distance from CGC (centroid of composite nontransformed section) to bottom fiber of composite section (i.e., to bottom of PDP)

$$y_{bc} = \frac{\left(A_g \bullet \left(\frac{h}{2}\right)\right) + \left(A_t \bullet \left(h + \frac{t_s}{2}\right)\right)}{A_c} = 3.866 \text{ in.}$$

y_{tg} = distance from CGC to top fiber of PDP (positive value means CGC is within CIP deck)

$y_{tg} = y_{bc} - h$ = 0.1164 in.

y_{tc} = distance from CGC to top fiber of composite section (i.e., to top of CIP slab)

$y_{tc} = h_{tot} - y_{bc}$ = 4.134 in.

I_c = moment of inertia of the composite nontransformed section

$$I_c = I_g + A_g \bullet \left(y_{bc} - \frac{h}{2}\right)^2 + I_t + A_t \bullet \left(h + \frac{t_s}{2} - y_{bc}\right)^2 = 478.33 \text{ in.}^4$$

S_{bc} = composite nontransformed section modulus for extreme bottom fiber (to bottom of PDP)

$$S_{bc} = \frac{I_c}{y_{bc}} = 123.7 \text{ in.}^3$$

S_{tg} = composite nontransformed section modulus for top fiber of PDP

$$S_{tg} = \frac{I_c}{y_{tg}} = 4110.8 \text{ in.}^3$$

S_{tc} = composite nontransformed section modulus for extreme top fiber (to top of CIP slab). Modular Ratio, n_{cip}, transforms section modulus back to CIP concrete so stress calculated at top of CIP corresponds to stress in CIP concrete (not in transformed PDP concrete)

$$S_{tc} = \left(\frac{1}{n_{cip}}\right)\left(\frac{I_c}{y_{tc}}\right) = 132.3 \text{ in.}^3$$

4.3 Noncomposite and Composite Transformed Section Properties

The transformed section refers to the section for which the strands (as well as the CIP concrete slab) are transformed to PDP concrete. This applies to both the noncomposite and composite sections.

Eccentricity of Strands, e_c

The distance between the center of gravity of the prestressing strands and the bottom fiber of PDP

$$y_{bs} = \frac{h}{2} = 1.875 \text{ in.}$$

The distance between the centroid of PDP and the bottom fiber of PDP

$$y_b = \frac{h}{2} = 1.875 \text{ in.}$$

Strand eccentricity, e_c (constant for straight strands)

$e_c = y_b - y_{bs}$ = 0 in.

Modular Ratio at Transfer, n_t, and at Final Time, n_f

For the single row of prestressing strands in the PDP, the steel area is multiplied by (n - 1) to transform the strands to PDP concrete, where n is the modular ratio (modulus of strand/modulus of PDP). Since E_c differs at transfer and at the final time, transformed section properties are calculated for these two stages.

	Strands to PDP		CIP to PDP
At Transfer:	$n_t = \dfrac{E_p}{E_{ci}} = 6.426$	$n_t - 1 = 5.426$	
At Final Time:	$n_f = \dfrac{E_p}{E_c} = 5.844$	$n_f - 1 = 4.844$	$n_{cip} = 0.875$

Centroid of Composite Section at Final Time

	Modulus of Elasticity	Area at Final Time
PDP Panel	$E_c = 4877$ ksi	$A_{PDP} = b\ h = 45$ in.^{2}
CIP Topping	$E_{cs} = 4266$ ksi	$A_{CIP} = n_{cip}\ b\ t_s = 44.61$ in.^{2}
PS Strands	$E_p = 28500$ ksi	$A_{ps\_tr} = (n_f - 1)\ A_{ps} = 0.823$ in.^{2}
		$A_{sum} = A_{PDP} + A_{CIP} + A_{ps\_tr} = 90.436$ in.^{2}

	y (from bottom to CG)	A y
PDP Panel	$y_{PDP} = \dfrac{h}{2} = 1.875$ in.	$Ay_{PDP} = A_{PDP}\ y_{PDP} = 84.38$ in.^{3}
CIP Topping	$y_{CIP} = h + \dfrac{(t_s)}{2} = 5.875$ in.	$Ay_{CIP} = A_{CIP}\ y_{CIP} = 262.1$ in.^{3}
PS Strands	$y_{ps} = \dfrac{h}{2} = 1.875$ in.	$Ay_{ps} = A_{ps\_tr}\ y_{ps} = 1.544$ in.^{3}
		$Ay_{sum} = Ay_{PDP} + Ay_{CIP} + A_{yps} = 348$ in.^{3}
		$y_{bar} = \dfrac{Ay_{sum}}{A_{sum}} = 3.8482$ in.

Overall centroid $y_{bar} = 3.8482$ in.

4.3.1 Noncomposite Transformed Section at Transfer

Noncomposite transformed section at transfer refers to PDP with strands transformed.

Area of transformed section at transfer, $A_{ti} = A_{PDP} + (n_t - 1)\, A_{ps} = 45.92$ in.^{2}

Moment of inertia of noncomposite transformed section at transfer, $I_{ti} = I_g = 52.73$ in.^{4}

Eccentricity of strands for noncomposite transformed section at transfer, $e_{ti} = 0$ in.

Distance from the centroid of the transformed section to the bottom fiber of PDP at transfer,

$$y_{bti} = y_{PDP} = 1.875 \text{ in.}$$

Section modulus for extreme bottom fiber of noncomposite transformed section at transfer

$$S_{bti} = \frac{I_{ti}}{y_{bti}} = 28.13 \text{ in.}^3$$

Section modulus for extreme top fiber of noncomposite transformed section at transfer

$$S_{tti} = \frac{I_{ti}}{h - y_{bti}} = 28.13 \text{ in.}^3$$

4.3.2 Noncomposite Transformed Section at Final Time

<u>Noncomposite Transformed Section at Final Time</u>

Area of transformed section at final time, $A_{tf} = A_{PDP} + A_{ps\_tr} = 45.82$ in.^{2}

Moment of inertia of the transformed section at final time, $I_{tf} = I_g = 52.73$ in.^{4}

Eccentricity of strands for noncomposite transformed section at final time, $e_{tf} = 0$ in.

Distance from the centroid of the noncomposite transformed section to the extreme bottom fiber of the beam at final time,

$$y_{btf} = y_{PDP} = 1.875 \text{ in.}$$

Section modulus for the extreme bottom fiber of the transformed section at final time

$$S_{btf} = \frac{I_{tf}}{y_{btf}} = 28.13 \text{ in.}^3$$

Section modulus for the extreme top fiber of the transformed section at final time

$$S_{ttf} = \frac{I_{tf}}{h - y_{btf}} = 28.13 \text{ in.}^3$$

4.3.3 Composite Transformed Section at Final Time

Moment of inertia, I_{tc}, for composite transformed section about overall centroidal axis

PDP Panel $$I_{PDP} = b \cdot \frac{h^3}{12} + A_{PDP} \cdot (y_{bar} - y_{PDP})^2 = 227.9 \text{ in.}^4$$

CIP Topping $$I_{CIP} = n_{cip} \cdot b \cdot \frac{t_s^{\,3}}{12} + A_{CIP} \cdot (y_{CIP} - y_{bar})^2 = 250.4 \text{ in.}^4$$

PS Strands $$I_{PS} = 0 \text{ in.}^4 + A_{ps\_tr} \cdot (y_{bar} - y_{ps})^2 = 3.206 \text{ in.}^4$$

Sum $$I_{tc} = I_{PDP} + I_{CIP} + I_{PS} = 481.57 \text{ in.}^4$$

Composite Transformed Section Properties

Area of composite transformed section at final time

$$A_{tc} = A_{sum} = 90.436 \text{ in.}^2$$

Moment of inertia of the composite transformed section at final time

$$I_{tc} = 481.57 \text{ in.}^4$$

Eccentricity of strands with respect to CGC of composite transformed section at final time

$$e_{tc} = y_{bar} - \frac{h}{2} = 1.973 \text{ in.}$$

Distance from CGC of composite transformed section to bottom fiber (of PDP)

$$y_{btc} = y_{bar} = 3.848 \text{ in.}$$

Section modulus for extreme bottom fiber of composite transformed section (in PDP)

$$S_{btc} = \frac{I_{tc}}{y_{btc}} = 125.14 \text{ in.}^3$$

Section modulus for top fiber of PDP

$$S_{ttc} = \frac{I_{tc}}{(h - y_{btc})} = -4903 \text{ in.}^3$$

NOTE: Since the CGC of the composite section is slightly above the PDP, this negative value is maintained to provide the correct stress at the top of the PDP (negative = T) in Section 6.3.2.2.

Section modulus for extreme top fiber of the transformed composite section (in CIP)

$$S_{dtc} = \frac{E_c}{E_{cs}} \cdot \frac{I_{tc}}{h_{tot} - y_{btc}} = 132.6 \text{ in.}^3$$

5.0 PRESTRESS LOSSES

Prestress losses in the strand stress consist of: 1) immediate losses (primarily elastic shortening at transfer, although elastic gains are considered due to strand extension under service loads), and 2) long-term losses due to concrete creep, concrete shrinkage, and steel relaxation.

For simplicity and clarity in demonstrating the calculation of immediate and long-term losses, the LRFD approach for Approximate Estimate of Time-Dependent Losses (LRFD 5.9.3.3) is used for long-term losses rather than the LRFD refined loss approach. The closed-form (direct) solution for elastic shortening is used per LRFD Eq. C5.9.3.2.3a-1, which eliminates iteration. Elastic gains are addressed for purposes of steel stress checks but are not needed in concrete stress checks because elastic gains (and elastic shortening) are implicitly accounted for when using transformed section properties.

Total loss in prestressing steel stress:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \qquad \text{[LRFD Eq. 5.9.3.1-1]}$$

Where:

Δf_{pT} = total loss (ksi)

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

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

5.1 Elastic Shortening Loss

The sum of losses due to elastic shortening, Δf_{pES}, at the time of application of prestress (ksi) is determined directly using the LRFD Commentary equation, LRFD Eq. C5.9.3.2.3a-1:

$$\Delta f_{pES} = \frac{A_{ps} \cdot f_{pbt} \cdot (I_g + e_m{}^2 \cdot A_g) - e_m \cdot M_g \cdot A_g}{A_{ps} \cdot (I_g + e_m{}^2 \cdot A_g) + A_g \cdot I_g \cdot \left(\frac{E_{ci}}{E_p}\right)} \qquad \text{[LRFD Eq. C5.9.3.2.3a-1]}$$

Where:

A_{ps} = area of prestressing strands (in.^{2})

A_g = gross area of section (in.^{2})

E_{ci} = Modulus of elasticity of PDP concrete at transfer (ksi)

E_p = Modulus of elasticity of prestressing steel (ksi)

e_m = average prestressing steel eccentricity at midspan (in.)

f_{pbt} = stress in prestressing steel immediately prior to transfer (ksi), taken equal to f_{pj}

I_g = moment of inertia of gross concrete section about centroidal axis (w/o reinforcement) (in.^{2})

M_g = midspan moment due to member self-weight (kip-in.)

Hence,

e_m = 0 in.

$M_g = M_{PDP}$ = 0.375 kip-ft = 4.5 kip-in.

$f_{pbt} = f_{pj}$ = 189 ksi

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

Δf_{pES} = 4.479 ksi

$$\text{Initial\_Loss} = \frac{\Delta f_{pES}}{f_{pj}} = 2.37\%$$

5.2 Approximate Estimate of Time-Dependent Losses

PDPs are taken, per LRFD 5.9.3.3, as "standard precast, pretensioned members subject to normal loading and environmental conditions where:

1) members are made from normal weight concrete;
2) the concrete is either steam- or moist-cured;
3) prestressing is by bars or strands with low relaxation properties; and
4) average exposure conditions and temperatures characterize the site." In this case, the long-term prestress loss, Δf_{pLT}, due to creep of concrete, shrinkage of concrete, and relaxation of steel can be estimated using LRFD Eq. 5.9.3.3-1, as shown below.

$$\Delta f_{pLT} = \frac{10.0 f_{pi} \bullet A_{ps}}{A_g} \bullet \gamma_h \bullet \gamma_{st} + 12.0 \bullet \gamma_h \bullet \gamma_{st} + \Delta f_{pR}$$ [LRFD Eq. 5.9.3.3-1]

Long-term prestress loss due to creep of concrete, shrinkage of concrete, and relaxation of steel (ksi),

In which:

$\gamma_h = 1.7 - 0.01H$ [LRFD Eq. 5.9.3.3-2]

$$\gamma_{st} = \frac{5}{(1 + f'_{ci})}$$ [LRFD Eq. 5.9.3.3-3]

Where:

f_{pi} = prestressing steel stress immediately prior to transfer (ksi), taken equal to f_{pj}

A_{ps} = area of prestressing strands (in.^{2})

A_g = gross area of PDP section (in.^{2})

H = average annual ambient relative humidity (percent), assumed as 70%

γ_h = correction factor for relative humidity of the ambient air

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

Δf_{pR} = an estimate of relaxation loss, taken as 2.4 ksi for low relaxation strand (ksi)

$$\Delta f_{pLT} = \frac{10.0 f_{pi} \cdot A_{ps}}{A_g} \cdot \gamma_h \cdot \gamma_{st} + 12.0 \cdot \gamma_h \cdot \gamma_{st} + \Delta f_{pR}$$

$f_{pi} = f_{pj} = 189$ ksi $\quad A_{ps} = 0.17$ in.^{2} $\quad A_g = 45$ in.^{2}

H = 70 $\quad \gamma_h = 1.7 - 0.01H$ $\quad \gamma_h = 1$

$$\gamma_{st} = \frac{5}{(1+f'_{ci})} = 0.909$$

$\Delta f_{pR} = 2.4$ ksi

$$\Delta f_{pLT} = \frac{10.0 f_{pi} \cdot A_{ps}}{A_g} \cdot \gamma_h \cdot \gamma_{st} + 12.0 \cdot \gamma_h \cdot \gamma_{st} + \Delta f_{pR}$$

$$\frac{10.0 f_{pi} \cdot A_{ps}}{A_g} \cdot \gamma_h \cdot \gamma_{st} = 6.491 \text{ ksi}$$

$12.0\ \gamma_h\ \gamma_{st} = 10.91$ ksi

$\Delta f_{pR} = 2.4$ ksi

$$\Delta f_{pLT} = \frac{10.0 f_{pi} \cdot A_{ps}}{A_g} \cdot \gamma_h \cdot \gamma_{st} + 12.0 \cdot \gamma_h \cdot \gamma_{st} + \Delta f_{pR} = 19.8 \text{ ksi}$$

$\Delta f_{pLT} = 19.8$ ksi

$$\text{Long\_term\_Loss} = \frac{\Delta f_{pLT}}{f_{pj}} = 10.48\%$$

5.3 Initial Losses, Strand Stress, and Strand Force at Transfer

Immediately after transfer, only elastic shortening, Δf_{pES}, is assumed to have occurred. The minor relaxation losses from jacking to transfer are ignored.

Per LRFD C5.9.3.2.3a: When calculating concrete stresses using transformed section properties, the effects of losses and gains due to elastic deformations are implicitly accounted for, Δf_{pES} should not be included in the prestressing force applied to the transformed section at transfer. (The PS strand and concrete are treated together as a composite section equally strained by PS force conceived as a fictitious external load.) Thus, the prestress force for jacking would be used in the PDP concrete stress analysis immediately after transfer. However, to determine the effective stress in the strands for LRFD strand stress checks, Δf_{pES} should be deducted from jacking stress to determine the actual strand stress immediately after transfer for the code check. The following calculations show relevant values.

1. For Concrete Stress Checks: PS Force just prior to transfer, P_{pi}, which is assumed to be equal to the Jacking force, P_{jack}:

$$P_{jack} = f_{pj}\, A_{ps} = 32.13 \text{ kip}$$

$$P_{pi} = P_{jack} = 32.13 \text{ kip}$$

As stated above, P_{pi} is the force to be used to check concrete stresses immediately after transfer, since transformed section properties are used in the concrete stress check.

2. For Steel Strand Checks: Effective stress in strands, f_{pt}, immediately after transfer

$$f_{pj} = 189 \text{ ksi} \qquad \Delta f_{pi} = \Delta f_{pES} = 4.479 \text{ ksi}$$

$$f_{pt} = f_{pj} - \Delta f_{pi} = 184.5 \text{ ksi}$$

5.4 Total Losses, Strand Stress, and Strand Force at Service

At service level, the total strand stress loss, Δf_{pT}, per LRFD Eq. 5.9.3.3-1, is the sum of the immediate losses (taken as elastic shortening only) plus the long-term losses (taken as the Approximate Estimate). However, for concrete stress checks, only long-term losses are deducted from the jacking stress, not the elastic shortening loss or elastic gain because of the use of the transformed section, as explained above).

$$\Delta f_{pES} = 4.479 \text{ ksi}$$

$$\Delta f_{pLT} = 19.8 \text{ ksi}$$

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} = 24.28 \text{ ksi}$$

$$\text{Total\_Loss} = \frac{\Delta f_{pT}}{f_{pj}} = 12.85\ \%$$

1. For Concrete Stress Checks: PS Force at Service Level should include time-dependent losses but not elastic shortening losses (or elastic gains at service) because transformed section properties are used:

Strand stress at service for concrete stress check, f_{pe}, based on time-dependent losses only:

$$f_{pe} = (f_{pj} - \Delta f_{pLT}) = 169.2 \text{ ksi}$$

PS Force at service, P_{pe} based on time-dependent losses only:

$P_{pe} = f_{pe} A_{ps} = 28.76$ kip

2. For Steel Strand Checks: (Actual) effective stress in strands, f_{pe_actual}, at Service Level

At service level, for strand checks, the total loss, Δf_{pT}, should be deducted from the jacking stress. This should include both elastic shortening and elastic gain, as shown below.

Calculate increase in strand stress due to Elastic Gain, Δf_{pEG}, at service level (due to deck weight, superimposed dead load, and live load). Also determine Elastic Gain due to DL. This example uses a 1.0 factor on Live Load, as explained previously.

$$\Delta f_{pEG} = \left(M_{CIP} . \frac{e_{tf}}{I_{tf}} + \left(M_b + M_{ws} \right) . \frac{e_{tc}}{I_{tc}} \right) . \frac{E_p}{E_c} + 1.0 . \left(M_{LL} . \frac{e_{tc}}{I_{tc}} \right) . \frac{E_p}{E_c} = 1.97 \text{ ksi}$$

$$\Delta f_{pEG\_DL} = \left(M_{CIP} . \frac{e_{tf}}{I_{tf}} + \left(M_b + M_{ws} \right) . \frac{e_{tc}}{I_{tc}} \right) . \frac{E_p}{E_c} = 0.206 \text{ ksi}$$

Actual effective stress in strands after all losses and all gains

$f_{pe\_actual} = f_{pj} - \Delta f_{pT} + \Delta f_{pEG} = 166.7$ ksi

Check PS steel stress limit at service limit state

$f_{pe\_actual} = 166.7$ ksi

$0.8 f_{py} = 194.4$ ksi [LRFD Table 5.9.2.2-1]

$f_{pe\_actual} < 0.8 f_{py}$ OK

Actual effective stress in strands after all losses and permanent gains:

$f_{pe\_actual\_DLgain} = f_{pj} - \Delta f_{pT} + \Delta f_{pEG\_DL} = 164.9$ ksi

Final Loss Percentage: $(\Delta f_{pT} - \Delta f_{pEG\_DL})/f_{pj} = 12.74$ %

6.0 CONCRETE STRESSES IN PDP SYSTEM

6.1 Concrete Stresses in PDP at Transfer

Concrete stress checks are based on transformed section properties. Therefore, PS force corresponds to jacking force since elastic shortening losses are implicitly accounted for.

Immediately after transfer, PS Force per ft, $P_{pi} = 32.13$ kip

6.1.1 Stress Limits for Concrete [LRFD Art. 5.9.2.3]

Compression (C): $f_{ci\_limit} = 0.65 f'_{ci} = 2.925$ ksi [LRFD Art. 5.9.2.3.1a]

Tension (T): PDPs have zero strand eccentricity (CGS coincides with PDP CGC). Thus, tension at the top of the PDP at transfer is not of concern. For completeness, a tension limit is shown and checked:

Without bonded reinforcement, the tension limit is:

$$-0.0948\sqrt{f'_{ci}} = -0.2011 \text{ ksi}$$

Not to exceed -0.2 ksi in tension [LRFD Tab. 5.9.2.3.1b-1]

$$f_{ti\_limit} = \text{if}\left(-0.0948\sqrt{f'_{ci}} \le -0.20, -0.20, -0.0948\sqrt{f'_{ci}}\right) = -0.2 \text{ ksi}$$

6.1.2 Stress at Midspan

Because the PDP strand has zero eccentricity, the midspan section is critical and should be checked.

Bending moment due to weight of the panel, per ft: M_{PDP} = 0.375 kip-ft

Top fiber stress in PDP

$$f_{t\_i} = \frac{P_{pi}}{A_{ti}} + \frac{M_{PDP}}{S_{tti}} = 0.86 \text{ ksi} \quad [C]$$

Stress Limit = 2.925 ksi since $f_{t\_i} \ge 0$

$$\text{D/C} = \frac{f_{t\_i}}{\text{StressLimit}} = 0.2939 \qquad \text{OK}$$

Bottom fiber stress in PDP

$$f_{b\_i} = \frac{P_{pi}}{A_{ti}} + \frac{M_{PDP}}{S_{bti}} = 0.54 \text{ ksi} \quad [C]$$

Stress Limit = 2.925 ksi since $f_{b\_i} \ge 0$

$$\text{D/C} = \frac{f_{b\_i}}{\text{StressLimit}} = 0.1845 \qquad \text{OK}$$

6.2 Concrete Stresses in PDP at Casting of Topping Slab

Based on the transformed section and approximate estimate of time-dependent losses, PS force per ft, P_{pe} = 28.76 kips.

6.2.1 Stress Limits for Concrete

LRFD Article 9.7.4.1 requires an unfactored stress check with the PDP concrete stresses limited to 0.65 f'_c (for compression) or the modulus of rupture, $0.24\sqrt{f'_c}$ (for tension)

For Service I Load Combination:

Compression (C): f_{c_limit} = 0.65 f'_c = 3.9 ksi

Tension (T): $f_{t\_limit} = -0.24\sqrt{f'_c} =$ -0.588 ksi [LRFD Art. 5.4.2.6]

6.2.2 Stresses at Midspan with Noncomposite Loads

Stresses are based on the bending moment due to PDP self weight, CIP topping, and a construction load of 50 psf, with all (service level) prestress losses.

Top fiber stress in PDP

$$f_{t\_cast} = \frac{P_{pe}}{A_{tf}} + \frac{(M_{PDP} + M_{CIP} + M_{const})}{S_{ttf}} = 1.14 \text{ ksi} \quad [C]$$

Stress Limit = 3.9 ksi since $f_{t\_cast} \geq 0$

$$\text{D/C} = \frac{f_{t\_cast}}{\text{StressLimit}} = 0.2922 \qquad \text{OK}$$

Bottom fiber stress in PDP

$$f_{b\_cast} = \frac{P_{pe}}{A_{tf}} - \frac{(M_{PDP} + M_{CIP} + M_{const})}{S_{btf}} = 0.116 \text{ ksi} \quad [C]$$

Stress Limit = 3.9 ksi since $f_{b\_cast} \geq 0$

$$\text{D/C} = \frac{f_{b\_cast}}{\text{Stress Limit}} = 0.0297 \qquad \text{OK}$$

6.2.3 Elastic Deformation [LRFD Art. 9.7.4.1]

LRFD Article 9.7.4.1 states that for PDP panels spanning less than 10 ft, the elastic deformation due to the dead load of the PDP plus the CIP topping should not exceed the panel span divided by 180 or 0.50 inches. The intent is to prevent excessive sagging of the formwork (PDP) during construction.

Elastic Deformation, Defl: $$\text{Defl} = \frac{5}{48}.(M_{PDP} + M_{CIP}).\frac{l^2}{E_c.I_g} = 0.036 \text{ in.}$$

Deflection Limit: $\text{Defl\_Limit} = \frac{l}{180} = 0.533 \text{ in.}$

$\text{Defl} \le 0.5 \text{ in. and Defl\_Limit} \le \frac{l}{180}$ OK

$$\text{D/C} = \frac{\text{Defl}}{\text{Defl\_Limit}} = 0.0672$$

6.3 Concrete Stresses in PDP at Service Loads

Using transformed section properties and approximate estimate of PS losses, PS Force per ft

P_{pe} = 28.76 kip

6.3.1 Stress Limits for Concrete

Compression (C) for Service I Load Combination: [LRFD Tab. 5.9.2.3.2a-1]

Permanent loads (PDP self weight, CIP slab, wearing surface, and barriers): 0.45 f'_c

For PDP: $f_{c\_limit\_DL} = 0.45\ f'_c = 2.7$ ksi

For CIP slab: $f_{cs\_limit\_DL} = 0.45\ f'_{cs} = 1.8$ ksi

Permanent and transient loads (all dead and live loads): 0.60 f'_c

For PDP: $f_{c\_limit\_TOT} = 0.60\ f'_c = 3.6$ ksi

For CIP slab: $f_{cs\_limit\_TOT} = 0.60\ f'_{cs} = 2.4$ ksi

Tension (T) for PS and permanent: 0 ksi ("no tension" allowed) [CA Amendments Tab. 5.9.2.3.2b-1]

Tension (T) for Service III load combination: $-0.19\sqrt{f'_c}$ Not to exceed – 0.6 ksi (T) [LRFD Tab. 5.9.2.3.2b-1]

For PDP:

$f_{t\_limit\_S3}$ = – 0.4654 ksi > – 0.6 ksi OK

6.3.2 Stresses at Midspan Due to Permanent and Transient (Live) Loads

The weight of the PDP and CIP slab act on the noncomposite section, producing midspan moments:

M_{PDP} = 0.375 kip-ft M_{CIP} = 0.425 kip-ft

At opening to traffic, the wearing surface, barriers, and live loads act on the composite section:

M_{ws} = 0.143 kip-ft M_{LL} = 6.14 kip-ft M_b = 0.574 kip-ft

6.3.2.1 Concrete Stress at Top Fiber of CIP Topping

Permanent Loads Only, Service I (only wearing surface and barrier produce perm stress in CIP deck)

$$f_{t\_CIP\_DL} = \frac{(M_{ws} + M_b)}{S_{dtc}} = 0.06489 \text{ ksi [C]}$$

Stress Limit = 0.45 f'_{cs} = 1.8 ksi

$$\text{D/C} = \frac{f_{t\_CIP\_DL}}{\text{Stress Limit}} = 0.036$$

Permanent and Transient Loads, Service I (total loads including live load)

$$f_{t\_CIP\_TOT} = \frac{(M_{ws} + M_b + M_{LL})}{S_{dtc}} = 0.621 \text{ ksi} \quad \text{[C]}$$

Stress Limit = 0.60 f'_{cs} = 2.4 ksi

$$\text{D/C} = \frac{f_{t\_CIP\_TOT}}{\text{Stress Limit}} = 0.2586$$

6.3.2.2 Concrete Stress at Top Fiber of the PDP Panel

Permanent Loads, Service I

$$f_{t\_PDP\_DL} = \frac{P_{pe}}{A_{tf}} + \frac{(M_{PDP} + M_{CIP})}{S_{ttf}} + \frac{(M_{ws} + M_b)}{S_{ttc}} = 0.967 \text{ ksi} \quad \text{[C]}$$

Stress Limit = 0.45 f'_c = 2.7 ksi

$$\text{D/C} = \frac{f_{t\_PDP\_DL}}{\text{Stress Limit}} = 0.3583$$

Permanent and Transient Loads, Service I

$$f_{t\_PDP\_TOT} = \frac{P_{pe}}{A_{tf}} + \frac{(M_{PDP} + M_{CIP})}{S_{ttf}} + \frac{(M_{ws} + M_b + M_{LL})}{S_{ttc}} = 0.952 \text{ ksi [C]}$$

Stress Limit = 0.6 f'_c = 3.6 ksi since $f_{t\_PDP\_TOT}$ >0

$$\text{D/C} = \frac{f_{t\_PDP\_TOT}}{\text{Stress Limit}} = 0.2645$$

6.3.2.3 Concrete Stress at Bottom Fiber of PDP

Permanent Loads, Service III

$$f_{b\_PDP\_DL} = \frac{P_{pe}}{A_{tf}} - \frac{(M_{PDP} + M_{CIP})}{S_{btf}} - \frac{(M_{ws} + M_b)}{S_{btc}} = 0.2176 \text{ ksi} \qquad [C]$$

Check 1: Per LRFD Tab. 5.9.2.3.2a-1 Allowable Compressive Stress (Compression Governs)

Stress Limit = 0.45 f'_c = 2.7 ksi

$$\text{D/C} = \frac{f_{b\_PDP\_DL}}{\text{Stress Limit}} = 0.0806$$

Check 2: Per CA Amendment Tab. 5.9.2.3.2b-1 "No Tension"

$f_{b\_PDP\_DL} \geq 0$ OK

Permanent and Transient Loads, Service III (LL factor of 1.0)

$$f_{b\_PDP\_TOT} = \frac{P_{pe}}{A_{tf}} - \frac{(M_{PDP} + M_{CIP})}{S_{btf}} - \frac{1.0(M_{ws} + M_b + M_{LL})}{S_{btc}} = -0.3712 \text{ ksi} \qquad [T]$$

Since $f_{b\_PDP\_TOT} < 0$

$$\text{Stress Limit} = -0.19\sqrt{f'_c} = -0.4654 \text{ ksi}$$

$$\text{DC} = \frac{f_{b\_PDP\_TOT}}{\text{Stress Limit}} = 0.7975$$ **NOTE: This is Governing D/C Ratio**

Since this stage often governs the PDP design, a breakdown of stress components is shown below:

Stress Components:

$$\frac{P_{pe}}{A_{tf}} = 0.628 \text{ ksi}$$

$$-\frac{(M_{PDP} + M_{CIP})}{S_{btf}} = -0.341$$

$$-\frac{(M_{ws} + M_b + M_{LL})}{S_{btc}} = -0.658$$

7.0 FLEXURAL STRENGTH CHECKS AT MIDSPAN

7.1 Flexural Strength Check for PDP System

7.1.1 Ultimate Moment for Strength I, M_u

M_u = 1.25 M_{DC} + 1.5 M_{DW} + 1.75 M_{LL}

DC: M_{PDP} = 0.375 kip-ft M_{CIP} = 0.425 kip-ft M_b= 0.574 kip-ft

DW: M_{ws} = 0.143 kip-ft

LL+IM: M_{LL} = 6.14 kip-ft

M_u = 1.25 (M_{PDP} + M_{CIP} + M_b) + 1.5(M_{ws}) + 1.75 (M_{LL}) = 12.68 kip-ft

7.1.2 Factored Flexural Resistance, $M_r = \phi M_n$

Average stress in prestressing strand, f_{ps}, when $f_{pe} \geq 0.5 f_{pu}$:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \quad \text{[LRFD Eq. 5.6.3.1.1-1]}$$

Where:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 0.28 \quad \text{[LRFD Eq. 5.6.3.1.1-2]}$$

k = 0.28 for low-relaxation strands in LRFD Table. C5.6.3.1.1-1 also.

d_p= distance from extreme compression fiber of composite section to centroid of prestressing steel

$$d_p = (h + t_s) - \frac{h}{2} = 6.125 \text{ in.}$$

c = distance from extreme compression fiber to neutral axis

To compute c, assume rectangular section behavior and then check if the depth of equivalent compression stress block, a, is less than or equal to t_s (CIP thickness), per LRFD Art. 5.6.3.2.3.

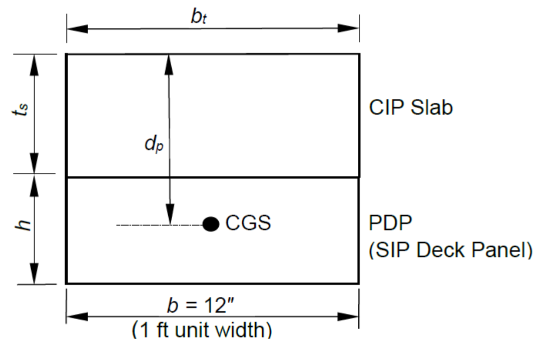


Figure 7.1.2-1 Section at Ultimate

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A'_s f'_y}{\alpha_1 f'_c \beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}} \quad \text{[LRFD Eq. 5.6.3.1.1-4]}$$

Where:

a = depth of equivalent rectangular stress block = $\beta_1 c$

b = effective width of compression flange: $b = w = 12$ in.

A_{ps} = area of prestressing steel: $A_{ps} = 0.17$ in.^{2}

A_s = area of mild steel tension reinforcement: $A_s = 0$ in.^{2}

A'_s = area of compression reinforcement: $A'_s = 0$ in.^{2}

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

f_y = yield strength of nonprestressed tension reinforcement: $f_y = 60$ ksi

f'_y = yield strength of nonprestressed compression reinforcement: $f'_y = 60$ ksi

f'_{cs} = compressive strength of slab concrete: $f'_{cs} = 4$ ksi

α_1 = stress block factor specified in Article 5.6.2.2, $\alpha_1 = 0.85$ for $f'_{cs} \leq 10$ ksi

β_1 = stress factor of compression block: [LRFD Art. 5.6.2.2]

β_1 = 0.85 since $f'_{cs} = 4$ ksi

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A'_s f'_y}{\alpha_1 f'_{cs}\beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}} = 1.248 \text{ in.}$$

$$a = \beta_1 c = 1.061 \text{ in.}$$

Verify assumption of rectangular section behavior, with stress block within CIP topping slab

$a \leq t_s$ OK

Verify $f_{pe} \geq 0.5\, f_{pu}$ to use general f_{ps} equation - OK

$$f_{ps\_gen} = f_{pu}\left(1 - k\frac{c}{d_p}\right) = 254.6 \text{ ksi}$$

However, the stress in PS strand due to <u>available development length</u>, l_d, (f_{ps_ld}) must also be checked as shown below, as this may be smaller than f_{ps} per LRFD Eq. 5.6.3.1.1-1.

$$l_d = K\left(f_{ps\_ld} - \frac{2}{3}f_{pe}\right) d_b$$ [LRFD Eq. 5.9.4.3.2-1]

which can be solved for f_{ps_ld} as follows:

$$f_{ps\_ld} = \frac{l_d}{Kd_b} + \frac{2}{3}f_{pe}$$

Where:

K = 1.0 for pretensioned panels with a depth less than 24 in. [LRFD 5.9.4.3.2]

d_b = nominal strand diameter = 0.375 in.

f_{pe} = effective stress in prestressing strands after all losses: f_{pe} = 169.2 ksi

Available development length at midspan of the PDP: $l_d = l/2$ = 4 ft

Stress in strand according to available development length, l_d:

$$f_{ps\_ld} = \frac{l_d}{Kd_b} + \frac{2}{3} f_{pe} = 240.8 \text{ ksi}$$

$$f_{ps} = \min\left(f_{ps\_gen},\ f_{ps\_ld}\right) = 240.8 \text{ ksi}$$

<u>Nominal Flexural Resistance,</u> M_n

M_n = nominal flexural resistance [LRFD Art. 5.6.3.2.1]

$M_n = A_{ps} f_{ps} \left(d_p - \frac{a}{2} \right)$ simplified for A_{ps} only [LRFD Eq. 5.6.3.2.2-1]

$$M_n = A_{ps} f_{ps} \left(d_p - \frac{a}{2} \right) = 19.09 \text{ kip-ft}$$

<u>Factored Flexural Resistance,</u> $M_r = \phi M_n$

Flexural resistance factor, ϕ, based on the net tensile strain in the PS steel at nominal resistance:

$$\phi = \min\left(0.75 + 0.25 \left(\frac{\varepsilon_t - \varepsilon_{cl}}{\varepsilon_{tl} - \varepsilon_{cl}} \right), 1.0 \right) \qquad 0.75 \le \phi \le 1.0 \qquad \text{[LRFD Eq. 5.5.4.2-1]}$$

(Note: lower limit of 0.75 will not govern)

Where:

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

$$\varepsilon_t = 0.003 \left(\frac{d_p - c}{c} \right) = 0.011723 \qquad \text{[LRFD Art. 5.6.2.1]}$$

ε_{cl} = compression-controlled strain limit in the extreme tension steel (in./in.)

$\varepsilon_{cl} = 0.002$ [LRFD Art. 5.6.2.1]

ε_{tl} = tension-controlled strain limit in the extreme tension steel (in./in.)

$\varepsilon_{tl} = 0.005$ [LRFD Art. 5.6.2.1]

$$\phi = \min\left(0.75 + 0.25\left(\frac{\varepsilon_t - \varepsilon_{cl}}{\varepsilon_{tl} - \varepsilon_{cl}}\right), 1.0\right) = 1.0$$ $0.75 \le \phi \le 1.0$ [LRFD Eq. 5.5.4.2-1]

Check of $\frac{c}{d_p}$ limit for viability of using Flexural Resistance Equations of 5.6.3.1 and 5.6.3.2 (vs. Strain Compatibility Equations)

$$\frac{c}{d_p} = 0.2038 \qquad \left(\frac{0.003}{0.003 + \varepsilon_{cl}}\right) = 0.6$$

$$\frac{c}{d_p} \le \left(\frac{0.003}{0.003 + \varepsilon_{cl}}\right)$$ OK [LRFD Eq. 5.6.2.1-1]

Therefore, the factored flexural resistance, $M_r = \phi M_n$, [LRFD Art. 5.6.3.2.1-1]

$M_r = \phi M_n = 19.085$ kip-ft

Check Factored Flexural Resistance, $M_r = \phi M_n \ge M_u$

$M_u = 12.68$ kip-ft

$\phi M_n \ge M_u$ OK

$$\text{D/C} = \frac{M_u}{\phi M_n} = 0.6642$$

NOTE: Calculations were checked for flexure at midspan. It is possible that intermediate sections between the midspan and the supports may be more critical due to partial development of the strands. However, because of the low D/C or ($M_u/\phi M_n$) ratio, other sections are not checked here.

7.2 Limits of Reinforcement Check

7.2.1 Maximum Reinforcement [LRFD Art. 5.6.2.1]

The check of maximum reinforcement limits was removed from the LRFD Specifications in 2005. Adequate ductility of the flexural member (PDP) is ensured by evaluating whether the member can be classified as tension-controlled, as was checked in determining the ϕ factor.

7.2.2 Minimum Reinforcement [LRFD Art. 5.6.3.3]

At any section, the amount of prestressed and nonprestressed tensile reinforcement must be adequate to develop a factored flexural resistance, M_r, greater than or equal to the lesser of:

1) The cracking moment based on an elastic stress distribution and the modulus of rupture, and

2) 1.33 times the factored moment required by the applicable strength load combination.

Check at midspan:

$$M_{cr} = \gamma_3 \left((\gamma_1 f_r + \gamma_2 f_{cpe}) S_{btc} - M_{dnc} \left(\frac{S_{btc}}{S_{btf}} - 1 \right) \right)$$ [LRFD Eq. 5.6.3.3-1]

Where:

f_r = modulus of rupture of concrete [LRFD Art. 5.4.2.6]

$f_r = 0.24\sqrt{f'_c} = 0.588$ ksi

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

$$f_{cpe} = \frac{P_{pe}}{A_{tf}} + \frac{P_{pe} e_{tf}}{S_{btf}} = 0.628 \text{ ksi}$$

M_{dnc} = total unfactored noncomposite dead load moment acting on the noncomposite section

$M_{dnc} = M_{PDP} + M_{CIP} = 0.8$ kip-ft

S_{btc} = section modulus for extreme fiber of the transformed composite section where the tensile stress is caused by externally applied loads:

$S_{btc} = 125.1$ in.^{3}

S_{btf} = section modulus for extreme fiber of the transformed noncomposite section where the tensile stress is caused by externally applied loads:

$S_{btf} = 28.13$ in.^{3}

γ_1 = flexural cracking variability factor

= 1.2 for precast segmental structures

= 1.6 for all other concrete structures $\gamma_1 = 1.6$

γ_2 = prestress variability factor

= 1.1 for bonded tendons

= 1.0 for unbonded tendons $\gamma_2 = 1.1$

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

= 0.67 for A615, Grade 60 reinforcement

= 0.75 for A706, Grade 60 reinforcement

= 1.00 for prestressing steel $\gamma_3 = 1.0$

$$M_{cr} = \gamma_3 \left((\gamma_1 f_r + \gamma_2 f_{cpe}) S_{btc} - M_{dnc} \left(\frac{S_{btc}}{S_{btf}} - 1 \right) \right) = 14.25 \text{ kip-ft}$$

At midspan, the factored moment required by Strength 1 load combination is:

M_u = 12.68 kip-ft

Thus, 1.33 M_u = 16.86 kip-ft

Controlling M_{cr}: M_{cr} = min (1.33 M_u, M_{cr}) = 14.25 kip-ft

Minimum Reinforcement Check: $M_r \geq$ min (1.33 M_u, M_{cr})

M_r = 19.09 kip-ft $\geq$ min (1.33 M_u, M_{cr}) OK

$$\text{D/C} = \frac{\min(1.33M_u, M_{cr})}{M_r} = 0.7467$$

NOTE: LRFD requires that this criterion be met at every section.

7.3 Flexural Strength Check for PDP Under Construction Loads

Per LRFD Article 3.4.2.1, the PDP (prior to hardening of the CIP topping) is checked for construction loads in the Strength Load Combination I with a load factor not less than 1.5. The wet concrete deck and PDP are considered as DC loads.

7.3.1 Ultimate Moment for Strength I, Mu, under Construction Loads

M_{u_con} = 1.25 M_{DC} + 1.5 M_{const}

DC: M_{PDP} = 0.375 kip-ft M_{CIP} = 0.425 kip-ft

CONST: M_{const} = 0.4 kip-ft

M_{u_con} = 1.25 (M_{PDP} + M_{CIP}) + 1.5 M_{const} = 1.6 kip-ft

7.3.2 Factored Flexural Resistance, $M_{r\_con} = \phi_{con} M_{n\_con}$ [LRFD Art. 5.6.3.2.1-1]

Average stress in prestressing strand, f_{ps}, when $f_{pe} \geq 0.5\, f_{pu}$:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$$ [LRFD Eq. 5.6.3.1.1-1]

Where:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 0.28$$ [LRFD Eq. 5.6.3.1.1-2]

k = 0.28 for low-relaxation strands [LRFD Table. C5.6.3.1.1-1]

d_p = distance from extreme compression fiber of composite section to the centroid of prestressing steel

$$d_p = \frac{h}{2} = 1.875 \text{ in.}$$ slab not hardened at this stage

c = distance from extreme compression fiber to the neutral axis

To compute c, assume rectangular section behavior

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A'_s f'_y}{\alpha_1 f'_c \beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}}$$ [LRFD Eq. 5.6.3.1.1-4]

Where:

a = depth of equivalent rectangular stress block = $\beta_1 c$

b = effective width of compression flange: $b = w$ = 12 in.

A_{ps} = area of prestressing steel: A_{ps}= 0.17 in.^{2}

A_s = area of mild steel tension reinforcement: A_s = 0 in.^{2}

A'_s = area of compression reinforcement: A'_s = 0 in.^{2}

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

f_y = yield strength of nonprestressed tension reinforcement: f_y = 60 ksi

f'_y = yield strength of nonprestressed compression reinforcement: f'_y = 60 ksi

f'_c = compressive strength of slab concrete: f'_c = 6 ksi

α_1 = stress block factor specified in Article 5.6.2.2, α_1 = 0.85 for $f'_{cs} \leq 10$ ksi

β_1 = stress factor of compression block: [LRFD Art. 5.6.2.2]

β_1 = 0.85 -0.05 (f'_c – 4.0) = 0.75 since f'_c = 6 ksi

$$c = \frac{A_{ps}f_{pu} + A_s f_y - A'_s f'_y}{\alpha_1 f'_{cs}\beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}} = 0.87 \text{ in.}$$

$a = \beta_1 c = 0.653$ in.

Verify $f_{pe} \geq 0.5\, f_{pu}$ to use general f_{ps} equation OK

$$f_{ps\_gen} = f_{pu}\left(1 - k\frac{c}{d_p}\right) = 234.9 \text{ ksi}$$

However, the stress in PS strand due to available development length, l_d, (f_{ps_ld}) must also be checked as shown below, as this may be smaller than f_{ps} per LRFD Eq. 5.6.3.1.1-1.

$$l_d = K\left(f_{ps\_ld} - \frac{2}{3}f_{pe}\right)d_b$$ [LRFD Eq. 5.9.4.3.2-1]

which can be solved for f_{ps_ld} as follows:

$$f_{ps\_ld} = \frac{l_d}{Kd_b} + \frac{2}{3}f_{pe}$$

Where:

K = 1.0 for pretensioned panels with a depth less than 24 in. [LRFD 5.9.4.3.2]

d_b = nominal strand diameter

d_b = 0.375 in.

f_{pe} = effective stress in prestressing strands after all losses

f_{pe} = 169.2 ksi

Available development length at midspan of the PDP: $l_d = l/2 = 4$ ft

Stress in strand according to available development length, l_d:

$$f_{ps\_ld} = \frac{l_d}{Kd_b} + \frac{2}{3}f_{pe} = 240.8 \text{ ksi}$$

f_{ps} = min (f_{ps_gen}, f_{ps_ld}) = 234.9 ksi

Nominal Flexural Resistance, M_n

M_{n_con} = nominal flexural resistance [LRFD Art. 5.6.3.2.1]

$$M_{n\_con} = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right)$$ simplified for A_{ps} only [LRFD Eq. 5.6.3.2.2-1]

$$M_{n\_con} = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) = 5.15 \text{ kip-ft}$$

Factored Flexural Resistance, $M_{r\_con} = \phi_{con}M_{n\_con}$ [LRFD Art. 5.6.3.2.1-1]

Flexural resistance factor, ϕ_{con}, based on the net tensile strain in the PS steel at nominal resistance:

$$\phi_{con} = \max\left(0.75 + 0.25\left(\frac{\varepsilon_t - \varepsilon_{cl}}{\varepsilon_{tl} - \varepsilon_{cl}}\right), 1.0\right) \quad 0.75 \le \phi_{con} \le 1.0$$ [LRFD Eq. 5.5.4.2-1]

(Note: upper limit of 1.0 will not govern)

Where:

$\varepsilon_t =$ net tensile strain in the extreme tension steel at nominal resistance (in./in.)

$$\varepsilon_t = 0.003\left(\frac{d_p - c}{c}\right) = 0.003465$$ [LRFD Art. 5.6.2.1]

$\varepsilon_{cl} =$ compression-controlled strain limit in the extreme tension steel (in./in.)

$\varepsilon_{cl} =$ 0.002 [LRFD Art. 5.6.2.1]

$\varepsilon_{tl} =$ tension-controlled strain limit in the extreme tension steel (in./in.)

$\varepsilon_{tl} =$ 0.005 [LRFD Art. 5.6.2.1]

$$\phi_{con} = \max\left(0.75 + 0.25\left(\frac{\varepsilon_t - \varepsilon_{cl}}{\varepsilon_{tl} - \varepsilon_{cl}}\right), 0.75\right) = 0.87 \quad 0.75 \le \phi_{con} \le 1.0$$ [LRFD Eq. 5.5.4.2-1]

<u>Check of $\frac{c}{d_p}$ limit for viability of using Flexural Resistance Equations of 5.6.3.1 and 5.6.3.2 (vs. Strain Compatibility Equations)</u>

$$\frac{c}{d_p} = 0.464 \qquad \left(\frac{0.003}{0.003 + \varepsilon_{cl}}\right) = 0.6$$

$$\frac{c}{d_p} \le \left(\frac{0.003}{0.003 + \varepsilon_{cl}}\right) \qquad \text{OK}$$ [LRFD Eq. 5.6.2.1-1]

$$\text{D/C} = \frac{\frac{c}{d_p}}{0.6} = 0.773$$

Therefore, the factored flexural resistance, $M_{r\_con} = \phi_{con} M_{n\_con}$, [LRFD Art. 5.6.3.2.1-1]

$M_{r\_con} = \phi_{con} M_{n\_con} = 4.495$ kip-ft

Check Factored Flexural Resistance, $M_{r\_con} = \phi_{con} M_{n\_con} \geq M_{u\_con}$

$M_{u\_con} = 1.6$ kip-ft

$\phi_{con} M_{n\_con} \geq M_{u\_con}$ OK

$$\text{D/C} = \frac{M_{u\_con}}{\phi_{con} M_{n\_con}} = 0.356$$

8.0 SUMMARY OF KEY VALUES AND DEMAND/CAPACITY RATIOS

8.1 Prestress Losses and Prestress Force (Jacking, After Transfer, Service)

Prestress Losses

$\Delta f_{pES} = 4.479$ ksi $\quad \text{Initial\_Loss} = \dfrac{\Delta f_{pES}}{f_{pj}} = 2.37\%$

$\Delta f_{pLT} = 19.8$ ksi $\quad \text{Long\_term\_Loss} = \dfrac{\Delta f_{pLT}}{f_{pj}} = 10.48\%$

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} = 24.28$ ksi $\quad \text{Total\_Loss} = \dfrac{\Delta f_{pT}}{f_{pj}} = 12.85\%$

Strand Stress and Prestress Force for Concrete Stress Check

Jacking

$f_{pj} = 189$ ksi $\quad P_{jack} = f_{pj} A_{ps} = 32.13$ ksi

After Transfer

For concrete stress check immediately after the transfer, using transformed section properties, $P_{pi} = P_{jack}$: $\quad P_{pi} = 32.13$ ksi

Service

$f_{pe} = (f_{pj} - \Delta f_{pLT}) = 169.2$ ksi $\quad P_{pe} = f_{pe} A_{ps} = 28.76$ kip

8.2 PDP Concrete Stress Ratios at Transfer

Stress at Midspan

Top fiber stress in PDP

Stress Limit = f_{ci_limit} = 2.925 ksi since f_{t_i} > 0

$$\text{D/C} = \frac{f_{t\_i}}{\text{Stress Limit}} = 0.2939$$

Bottom fiber stress in PDP

Stress Limit = f_{ci_limit} = 2.925 ksi since f_{b_i} > 0

$$\text{D/C} = \frac{f_{b\_i}}{\text{Stress Limit}} = 0.1845$$

8.3 PDP Concrete Stress Ratios at Casting of Slab

Concrete Stresses in PDP at Casting of Topping Slab

Stresses at Midspan with Noncomposite Loads

Stresses are based on the bending moment due to PDP self weight, CIP topping, and a construction load of 50 psf, with all (service level) prestress losses.

Top fiber stress in PDP

Stress Limit = f_{c_limit} = 3.9 ksi since f_{t_cast} > 0

$$\text{D/C} = \frac{f_{t\_cast}}{\text{Stress Limit}} = 0.2922$$

Bottom fiber stress in PDP

Stress Limit = f_{c_limit} = 3.9 ksi since f_{b_cast} > 0

$$\text{D/C} = \frac{f_{b\_cast}}{\text{Stress Limit}} = 0.0297$$

Elastic Deformation

$$\text{Defl\_limit} = \frac{l}{180} = 0.5333 \text{ in.}$$

$$\text{D/C} = \frac{\text{Defl}}{\text{Defl\_limit}} = 0.0672$$

8.4 PDP Concrete Stress Ratios at Service Loads

Concrete Stresses in PDP at Service Loads

Stresses at Midspan due to Permanent and Transient (Live) Loads

Concrete Stress at Top Fiber of CIP Topping

Permanent Loads Only, Service I (only wearing surface and barrier produce perm stress in CIP deck)

Stress Limit = 0.45 f'_{cs} = 1.8 ksi $\quad D/C = \dfrac{f_{t\_CIP\_DL}}{\text{Stress Limit}} = 0.036$

Permanent and Transient Loads, Service I (total loads including live load)

Stress Limit = 0.60 f'_{cs} = 2.4 ksi $\quad D/C = \dfrac{f_{t\_CIP\_TOT}}{\text{Stress Limit}} = 0.2586$

<u>Concrete Stress at the Top Fiber of the PDP panel</u>

Permanent Loads, Service I

Stress Limit: 0.45 f'_c = 2.7 ksi >, since $f_{t\_PDP\_DL} > 0$ $\quad D/C = \dfrac{f_{t\_PDP\_DL}}{\text{Stress Limit}} = 0.3583$

Permanent and Transient Loads, Service I

Stress Limit: 0.6 f'_c = 3.6 ksi since $f_{b\_PDP\_DL} > 0$ $\quad D/C = \dfrac{f_{t\_PDP\_TOT}}{\text{Stress Limit}} = 0.2645$

<u>Concrete Stress at Bottom Fiber of PDP (Service III)</u>

Permanent Loads, Service III

Stress Limit: 0.45 f'_c = 2.7 ksi, since $f_{b\_PDP\_DL} > 0$ $\quad D/C = \dfrac{f_{b\_PDP\_DL}}{\text{Stress Limit}} = 0.0806$

Permanent and Transient Loads, Service III

Stress Limit: $-0.19\sqrt{f'_c} = -0.465$ ksi since $f_{b\_PDP\_TOT} < 0$

$$D/C = \frac{f_{b\_PDP\_TOT}}{\text{Stress Limit}} = 0.7975$$

NOTE: This is Governing D/C Ratio

8.5 Flexural Strength Ratios at Ultimate and Under Construction Loads

Flexural Strength Check for PDP System at Midspan

<u>PDP with CIP Deck at Ultimate</u>

<u>Check Factored Flexural Resistance:</u> $M_r = \phi M_n \geq M_u$

$\phi M_n > M_u$ OK $\quad D/C = \dfrac{M_u}{\phi M_n} = 0.6642$

<u>Minimum Reinforcement Check</u>: $M_r \geq \min(1.33M_u, M_{cr})$:

$$D/C = \frac{\min(1.33M_u, M_{cr})}{M_r} = 0.7467$$

<u>PDP under Construction Loads</u>

$$\text{D/C} = \frac{M_{u\_con}}{\phi_{con} M_{n\_con}} = 0.356$$

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