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Chapter Seventeen

PRESTRESSED CONCRETE SUPERSTRUCTURES

Section 5 of the **LRFD Bridge Design Specifications** unifies the design provisions for concrete reinforced with steel reinforcing bars and/or prestressing tendons. Chapter Seventeen presents MDT supplementary information specifically for fully prestressed concrete components. Chapter Sixteen discusses reinforced concrete components.

17.1 GENERAL

17.1.1 Definitions

Reference: LRFD Article 5.2

The following definitions apply:

1. Prestressing. Prestressing is the process of inducing stresses and deformations into a component prior to the application of the external loads. In the context of this section, high-strength steel is used to induce compressive stresses and deformations into a concrete component.
2. Pretensioning. Pretensioning is the process in which a concrete component is prestressed by releasing into it the force of steel tendons that are tensioned prior to concrete being cast around the tendons.
3. Posttensioning. Posttensioning is the process in which a concrete component is prestressed by tensioning steel tendons that are inserted into ducts cast through the component.
4. Partial Prestressing. Partial prestressing is the process of prestressing in which the concrete component is allowed to experience service-load tension; i.e., the concrete component resists loads through prestressed tendons and mild reinforcement.

17.1.2 Practices and Responsibilities

17.1.2.1 MDT Practices

MDT's prestressed concrete designs are typically limited to precast, fully prestressed girders. Therefore, the design provisions discussed herein address solely precast, prestressed girders.

Posttensioning or partial prestressing shall be used only with the approval of the Bridge Design Engineer.

17.1.2.2 Responsibilities

MDT's preliminary design process must ensure that the chosen standard girder can achieve the specified resistance. In other words, can the prestressing force be achieved within the cross section and the allowable stresses met. For MDT design applications, it is assumed that the girder can be fabricated in a single day with a released strength of 41.5 MPa.

The MDT preliminary design results in a specified ultimate moment capacity, termed the nominal resistance in the LRFD Specifications, and the allowable top and bottom stresses for the chosen girder section and girder spacing.

The fabricator is responsible for the final design of the prestressed concrete girder. Shop drawings and a complete set of final design calculations are submitted by the fabricator for Bridge Bureau approval.

The fabricator or general contractor is responsible for investigating stresses in the components during proposed handling, transportation and erection.

17.2 MATERIALS

17.2.1 Concrete

Reference: LRFD Article 5.4.2.1

The typical specified 28-day compressive strength for precast, prestressed girders is 48 MPa. The specified strength may be increased to 52 MPa with the approval of the Bridge Area Engineer.

17.2.2 Prestressing Tendons

17.2.2.1 Material Properties

Reference: LRFD Article 5.4.4.1

The prestressing tendons typically used for prestressed girders in Montana are low-relaxation, seven-wire strands conforming to AASHTO M203 (ASTM A416), Grade 1860. The tensile strength, f_{pu}, and yield strength, f_{py}, of this steel are 1860 MPa and 1675 MPa, respectively.

17.2.2.2 Geometric Properties

References: LRFD Articles 5.4.4.1 and 5.11.4.3

The following applies:

1. Diameter. Seven-wire strand of 12.7-mm diameter, with an area of 98.77 mm^2, are the typical tendons used in Montana.

2. Number. An even number of strands must be specified.

3. Spacing. The minimum spacing of seven-wire strand tendons shall be 50 mm center to center.

4. Trajectory. For simplicity of fabrication, straight trajectories are preferred. However, harped trajectories are more typical. Harped trajectories help to control stresses and camber, and they contribute to shear resistance. An alternative to harping strands is debonding or shielding strands. The LRFD limit of a maximum of 25% of the strands debonded shall be observed.

17.3 BRIDGE DETAILS

17.3.1 Continuity

Reference: LRFD Article 5.14.1.2.7

17.3.1.1 General

Multiple-span, prestressed concrete girder bridges consist of simple span girders supporting continuous cast-in-place, reinforced concrete decks. In some cases, the prestressed concrete girders are made continuous after erection to resist live load as continuous girders. In these cases, the stiffness of the cast-in-place diaphragms encasing the ends of the precast girders over the piers is neglected.

17.3.1.2 Slab Reinforcement

Reference: LRFD Articles 5.11.1.2.3 and 5.14.1.2.7b

If a bridge is designed so that the girders and slab act continuously over the bent, the longitudinal reinforcement in the slab over an internal pier shall be anchored in zones that are crack-free; i.e., in compression, at strength limit states. This means that the steel used to resist the negative moment must extend past the point of contraflexure. The embedment length into the compression zone shall satisfy LRFD Article 5.11.1.2.3. Further, the terminating bars shall be staggered.

This treatment should not be confused with the routine placement of S5 and S6 Bars (see the **MDT Standard Bridge Details** MSL-5 and MSL-6) as additional reinforcing steel in the slab over bents where only the slab is continuous.

17.3.2 Diaphragms

Reference: LRFD Article 5.13.2.2

17.3.2.1 General

Standard intermediate and end diaphragm details, as shown on **MDT Standard Bridge Details** MSL-5 and MSL-6, shall be used on all prestressed girder superstructures, where applicable.

17.3.2.2 Intermediate Diaphragms

Intermediate diaphragms shall be spaced as shown on Standard Drawings MB-1, MB-A, MB-4, MM-72 and MMT-28. All intermediate diaphragms shall be placed normal to the centerline of the girders.

17.3.2.3 End Diaphragms

End diaphragms shall be provided at all intermediate supports and abutments where expansion joints are located.

End diaphragms at intermediate supports shall be placed parallel to the skew of the abutment.

17.3.2.4 Diaphragms for Girders Made Continuous for Live Load

There are several possible treatments at the girder end to structurally engage the diaphragm and attain design continuity. Among the possible treatments are the following:

1. Serrate the girder end and extend the mild reinforcing steel for shear transfer to the diaphragm. Extend a portion of the prestressing strands from the girder bottom for positive moment capacity over the support caused by live loading in remote spans. The girder is temporarily supported on hardwood blocks until the diaphragm concrete has cured.

2. Set girder ends on thin elastomeric pads directly on the cap. Extend a portion of the prestressing strands from the girder bottom for positive moment capacity over the

support caused by live loading in remote spans. Place the diaphragm concrete around the girder ends. The diaphragm is the width of the cap below it.

Obtain the approval of the Bridge Area Engineer for the methods to be used prior to proceeding with design.

17.4 STANDARD GIRDERS

17.4.1 General

MDT has available for bridge design a series of prestressed, precast concrete girder sections. The sections to be used for design are those historically produced by local fabricators. Fabricators are allowed to supply girders with similar section properties provided that the girder design meets contract requirements.

General information on the available girder types and typical span ranges is presented in Chapter 13 of this **Manual**. More specific design information for prestressed girder bridges is presented in this Chapter.

One standard girder cross section shall be used exclusively in a structure, unless unusual circumstances dictate a need to vary girder section depth within a structure. An example might be a localized vertical clearance problem. Any intended use of different depth girders within a single structure must be approved by the Bridge Area Engineer in writing before the layout is finalized.

Girder lengths are usually rounded to the nearest 0.5-m increment at the conclusion of the bridge length calculations.

17.4.2 Prestressed I-Girder Section

Unless otherwise approved, simple span lengths for the various standard girders shall not exceed the centerline-to-centerline-bearing lengths in Figure 17.4A.

Girder Type	Span Length Limit (m)
Type 1	17
Type MT-28	23
Type A	26
Type IV	35
Type M-72	45

STANDARD GIRDER LENGTHS
Figure 17.4A

Slightly greater span lengths may be used for girders made continuous for live load. For I-Girder section properties, see Figure 17.4B

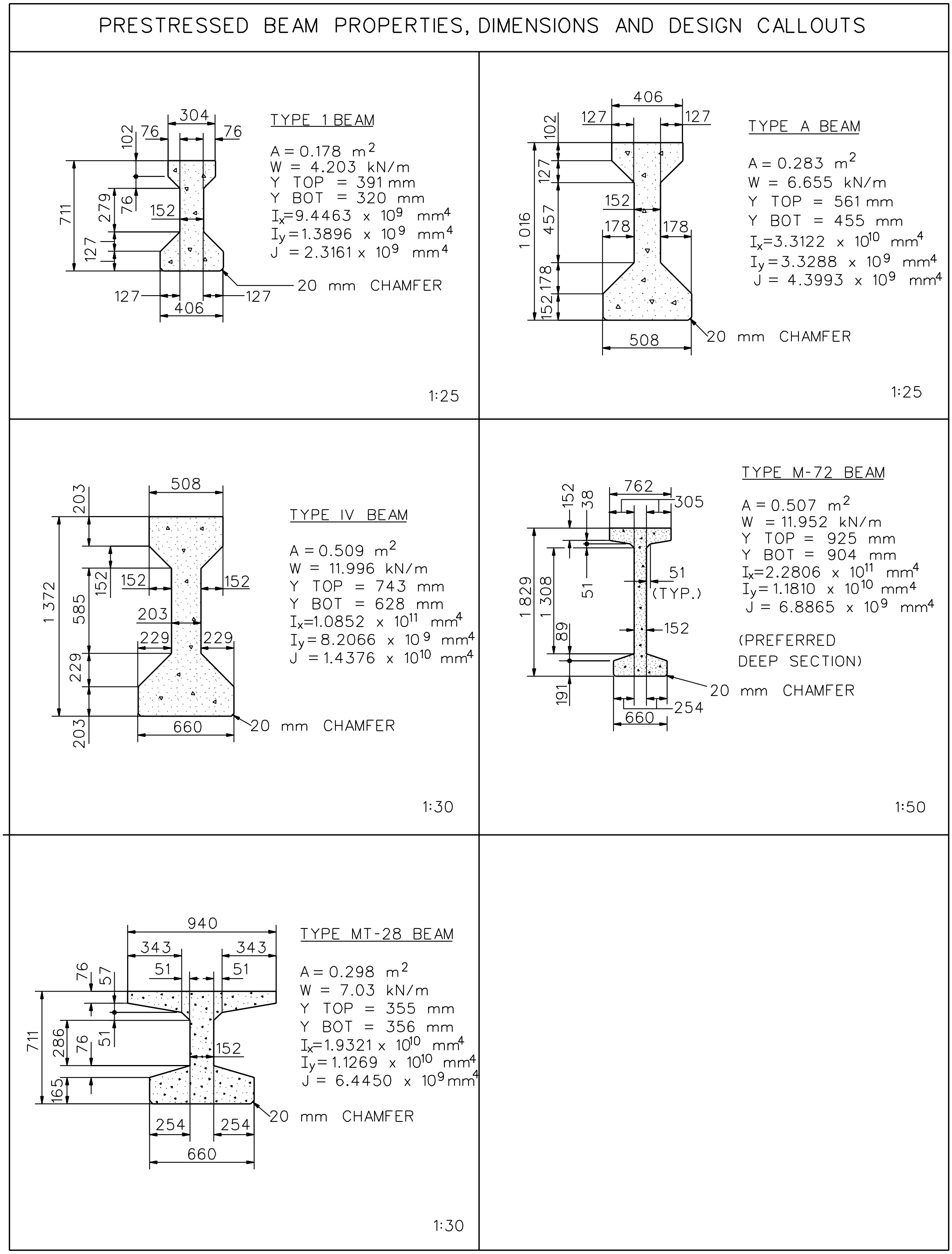
17.4.3 Joined Prestressed Precast Girder Sections

Also available are two types of joined prestressed, precast girder sections. These are bulb-T girders and tri-deck girders. These secions are placed side-by-side forming an instant roadway making them a good choice for remote locations where concrete availability is a problem or where reducing the duration of a road closure can save the cost of a detour. The jointed precast sections can be fabricated in a variety of depths and widths that allow these girders to be customized over a wide range of span lengths and roadway widths.

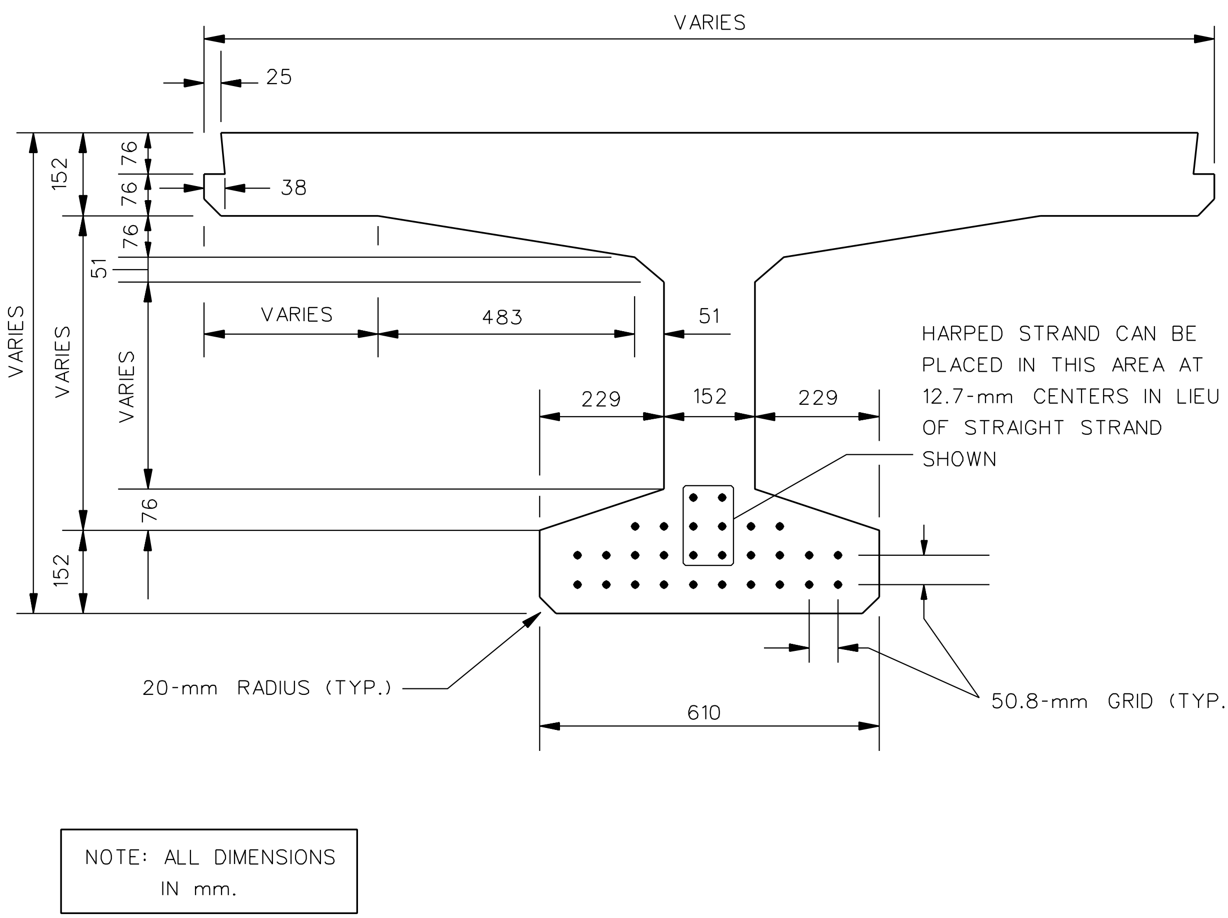
Figure 17.4C identifies the general shape of a bulb-T girder. Because of the wide variety of possible dimensions, no section properties have been included in this **Manual**. The designer should consult with local fabricators for the latest information to use in design calculations.

Bulb-T girders can be fabricated in a range of depths from 889 mm to 1422 mm, and the top flange can vary in width between 1219 mm and 2438 mm. The bottom flange dimensions and the web thickness are constant for all sections. This girder type allows for economical spans from about 14 m to about 40 m in length.

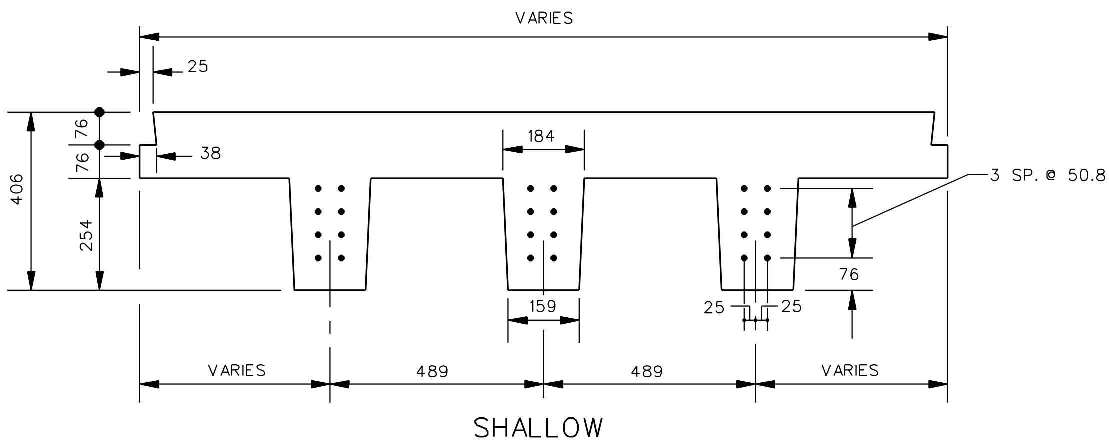
Figure 17.4D identifies the general shape of a tri-deck girder. Tri-deck girders come in two depths — 406 mm or 685 mm. The range of possible top flange widths is the same for both depths because they can be varied from 1212 mm to 1822 mm. This girder type allows for economical spans from approximately 10 m to 22 m in length. Because of their shallow depth, tri-deck girders are often considered as an option to cast-in-place flat slab bridges.

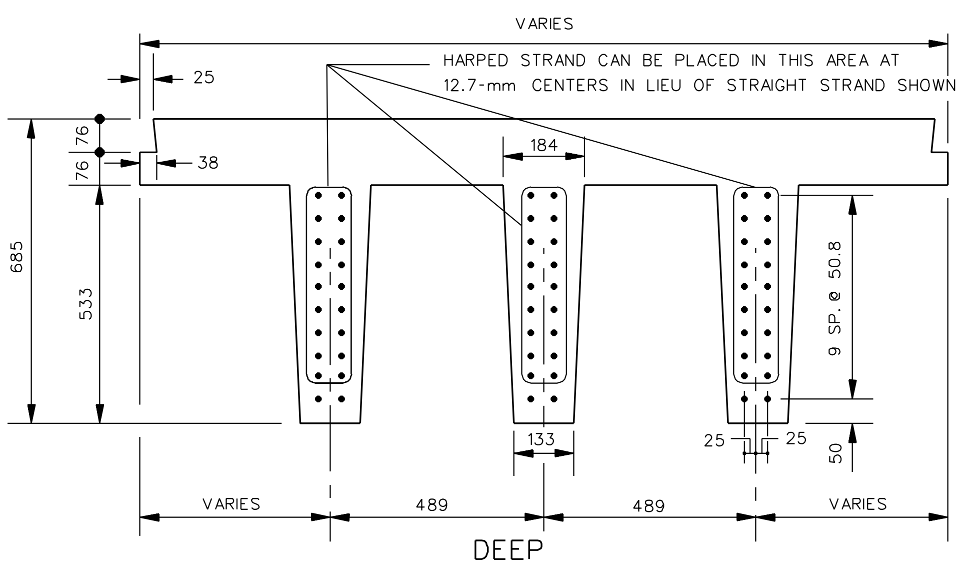


PROPERTIES OF STANDARD GIRDER SECTIONS
Figure 17.4B



BULB-T
Figure 17.4C





NOTE: ALL DIMENSIONS IN mm.

TRI-DECK
Figure 17.4D

17.5 GIRDER DESIGN

17.5.1 General

In general, bridge cross sections shall be selected which yield the minimum number of girders.

17.5.2 Prestressed Losses

Reference: LRFD Article 5.9.5

Time-dependent prestressed losses shall be taken as tabularized in LRFD Table 5.9.5.3-1 or as calculated as itemized values by the MDT computer program, Prestressed Beam Design.

17.5.3 Flexural Resistance

17.5.3.1 General

References: LRFD Articles 5.7.2.2, 5.7.3.1.1 and 5.7.3.2.2

The general LRFD equation for the nominal, flexural resistance of flanged concrete sections when applied to fully prestressed concrete sections reduces to the following:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + 0.85 f'_c\,(b - b_w)\beta_1 h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$$

where:

A_{ps} = area of prestressing steel

f_{ps} = average stress in prestressing steel at nominal flexural resistance specified in LRFD Article 5.7.3.1.1

d_p = distance from extreme compression fiber to the centroid of prestressing tendons

a = depth of the equivalent stress block

b = width of compression face of the member

b_w = web width

β_1 = stress block factor specified in LRFD Article 5.7.2.2

h_f = compression flange depth

17.5.3.2 Example

The end of Section 17.5 presents a typical flexural design of a simple span, Type M72 prestressed concrete girder with a continuous bridge deck.

17.5.4 Shear Resistance

Reference: LRFD Article 5.8

The drawings for the MDT Standard sections were developed by using an envelope of “worst-case” loadings for simple spans. In general, shear does not need to be checked on a project-specific basis if using standard girder sections. However, if the designer has a situation where shear needs to be investigated (e.g., girders made continuous for live load), the following discussion may be helpful.

17.5.4.1 Shear Resistance of Concrete

In the LRFD Specifications, the shear resistance of the concrete, V_c, shall be taken as the lesser of V_{ci} or V_{cw}:

$$V_{ci} = 0.6\sqrt{f'_c}\,b'd + V_d + \frac{V_i M_{cr}}{M_{max}}$$

but need not be less than:

$$1.7\sqrt{f'_c}\,b'd$$

and

$$V_{cw} = (3.5\sqrt{f'_c} + 0.3 f_{pc})b'd + V_p$$

where:

b' = width of the web of a flanged member

d = distance from extreme compressive fiber to centroid of the prestressing force

V_d = shear force at section due to unfactored dead load

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}

M_{cr} = moment causing flexural cracking at section due to externally applied loads

M_{max} = maximum factored moment at section due to externally applied load

f_{pc} = compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads

V_p = vertical component of effective prestress force at the section

The shear resistance of the concrete thus calculated shall be used in LRFD Equation 5.8.3.3-1.

17.5.4.2 Shear Resistance of Steel

Reference: LRFD Article 5.8.3.3

The general LRFD Equation for the shear resistance of transverse steel reinforcement, V_s, can be simplified for use with the traditional value of concrete shear resistance and vertical stirrups as follows:

$$V_s = \frac{A_s f_y d}{s}$$

17.5.5 Erection Plan Drawing Notes

Erection plan drawings have several entries that pertain to the design of the prestressed girders. A note that sets the maximum f_c' (compressive strength of concrete at 28 days) for design to 48 MPa is placed on the drawing.

A tabulation of design stresses at extreme fibers without prestress is made for girder dead load stresses and total dead load plus live load design stresses. The required factored moment in kN-m is also shown.

A typical beam-design-stresses table is shown in the example at the end of Section 17.5.

Prestressed Concrete Beam Example

Given: Type M-72 Girder:

A	=	$0.507\ m^2$
W	=	11.952 kN/m
y_{top}	=	925 mm
y_{bottom}	=	904 mm
I	=	$2.2806 \times 10^{11}\ mm^4$

Roadway Width	=	12 m (face to face of curbs)
Span Length	=	35 m
Girder Spacing	=	2650 mm (5-girder cross section)
Slab Thickness	=	200 mm
Haunch Depth	=	220 mm
Wearing Surface	=	35 mm
f'_c Girder	=	48 MPa (41.5 MPa @ transfer) (Section 17.1.2.2)
f'_c Slab	=	31 MPa

Interior Girder Design
HL-93 Live Load

Loads:

Dead Load Moments:

Girder: $$M = \frac{WL^2}{8} = \left(11.952\,\frac{kN}{m}\right)\frac{(35\ m)^2}{8} = 1830\ kN \bullet m \qquad \gamma = 2400\,\frac{kg}{m^3}$$

Slab: $$(2.65\ m)\left(\frac{200\ m}{1000}\right)\left(23.537\,\frac{kN}{m^3}\right)\left(\frac{(35\ m)^2}{8}\right) = 1910\ kN \bullet m$$

Fillet: $$\left(\frac{762\ mm}{1000}\right)\left(\frac{20\ mm}{1000}\right)(23.537)\left(\frac{35^2}{8}\right) = 55\ kN \bullet m$$

Barrier : $$2\left(\frac{5.187\ kN/m}{5\ girders}\right)\left(\frac{(35\ m)^2}{8}\right) = 318\ kN \bullet m$$

Diaphragm: $$M = P\frac{L}{3} = (24.55\ kN)\frac{35}{3} = 286\ \ kN \bullet m$$

Future Wearing Surface: $$\left(0.5\,\frac{kN}{m^2}\right)\left(\frac{12\ m}{5\ girders}\right)\left(\frac{(35\ m)^2}{8}\right) = 184\ kN \bullet m$$

Live Load Moment:

Interior girder, multiple lanes:

$$g = 0.075 + \left(\frac{S}{2900}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{k_g}{Lt_S^3}\right)^{0.1} \qquad \text{(LRFD Table 4.6.2.2.2b-1)}$$

where: $k_g = n(I + Ae_g^2)$

$$n = \frac{33\,200\text{ MPa}}{26\,700\text{ MPa}} = 1.243$$

$I = 2.2806 \text{ x } 10^{11}\text{ mm}^4$ (Figure 17.4B)

$A = 0.507\text{ m}^2$

$e_g = 925\text{ mm} + 20\text{ mm} + \dfrac{165}{2} = 1028\text{ mm}$

$$k_g = 1.24\,[2.28 \text{ x } 10^{11} + 0.507\,(1028)^2] = 9.46 \text{ x } 10^{11}\text{ mm}^4$$

$$g = 0.075 + \left[\frac{2650}{2900}\right]^{0.6}\left[\frac{2650}{35\,000}\right]^{0.2}\left[\frac{9.46 \text{ x } 10^{11}}{(35\,000)(165)^3}\right]^{0.1}$$

$$= 0.075 + (0.947)(0.597)(1.197)$$

$$g = 0.075 + 0.676 = 0.752$$

IM = 0.33 for truck and tandem

M_{LL+IM} = (4691.5 kN•m) 0.752 @ mid-span (LL + IM moment from BTBEAM)
= 3528 kN•m

Section Properties:

$$\frac{E_{slab}}{E_{girder}} = \frac{26\,700}{33\,200} = 0.804$$

$$t_{effective} = 200 - 35 = 165\text{ mm}$$

$$\frac{1}{4}(L) = \frac{1}{4}(35\text{ m}) = 8.75\text{ m}$$

$12(t_{effective}) + \dfrac{1}{2}(762) = 2361\text{ mm}$ (governing effective slab width)

$S = 2560\text{ mm}$

Location of composite section NA:

Area:

$$NA = \frac{\sum Ay}{\sum A}$$

		ΣA			ΣAy
0.804 (2361 mm)(165 mm)	=	313 210.30	• 1931.5	=	604 965 694.5
0.804 (762 mm) (20 mm)	=	12 252.96	• 1839	=	22 533 193.4
		507 000.00	• 904	=	458 328 000.0
		832 463.26			1 085 826 887.9

NA = 1304.4 from bottom = y_B

$y_T = (1829 + 20 + 165) - y_B = 709.6$ mm

Composite Section Modulus, I_{comp}: $I = I_0 + Ad^2$

Slab: $\dfrac{0.804(2361)(165)^3}{12} + 313\,210.3\,(627.1)^2$

= 710 595 777.4 + 123 171 331 732
= 123 881 927 509

Haunch: $\dfrac{(0.804)(762)(20)^3}{12} + 12\,252.96\,(534.6)^2$

= 408 432 + 3 501 861 169.6
= 3 502 269 601.6

Girder: $2.2806 \times 10^{11} + (507\,000)(400.4)^2$
$= 2.2806 \times 10^{11} + 81\,282\,321\,120$
$= 309\,342\,321\,120 \text{ mm}^4$

I_{comp}: $= 4.367 \times 10^{11} \text{ mm}^4$

Stresses: $\sigma = \dfrac{MC}{I}$

DC beam: $\left[\dfrac{(1830 \text{ kN} \bullet \text{m})(1000)(1000)}{2.2806 \times 10^{11} \text{ mm}^4}\right]$ x 925 mm = 7.422 MPa (Top)
x 904 mm = 7.254 MPa (Bottom)

Slab, fillet & diaphragm: $\left[\dfrac{(2251)(1000)(1000)}{2.2806 \times 10^{11}}\right]$ x 925 mm = 9.130 MPa (Top)
x 904 mm = 8.923 MPa Bottom)

Barrier and future wearing surface: $\left[\dfrac{(318+184)(1000)(1000)}{4.367 \times 10^{11}}\right]$ x 710 mm = 0.816 MPa (Top)
x 1304 mm = 1.499 MPa (Bottom)

LL + IM: $\left[\dfrac{(3528)(1000)(1000)}{4.367 \times 10^{11}}\right]$ x 710 mm = 5.736 MPa (Top)
x 1304 mm = 10.535 MPa (Bottom)

Design Stresses (Top): + 7.422 + 8.615 + 0.259 + 5.736 = +22.032 MPa

Design Stresses (Bottom): –7.254 + (–8.419) + (–0.475) + (–10.535) = –26.683 MPa

Design Stresses

Transfer: (Beam DL only)

$$\sigma_{t_{top}} = 7.42 \text{ MPa}$$

$$\sigma_{t_{bot}} = 7.25 \text{ MPa}$$

Final:

Compression Stresses:

$$\sigma_{f_{top}} = DC + DW = 7.42 + 9.13 + 0.82 = 17.37 \text{ MPa}$$
$$\sigma_{f_{top}} = 0.5\left(DC + DW\right) + LL + IM = 0.5\left(17.37\right) + 5.74 = 14.43 \text{ MPa}$$
$$\sigma_{f_{top}} = DC + DW + LL + IM = 17.37 + 5.74 = 23.11 \text{ MPa}$$

Tension Stresses:

$$\sigma_{f_{bot}} = DC + DW + 0.8\left(LL + IM\right) = -7.25 - 8.92 - 1.50 - 0.8(10.54) = -26.10 \text{ MPa}$$

Design Stress Table for Plans

BEAM DESIGN STRESSES		
Beam Length (C.L Brg. to C.L. Brg.) (Horizontal Distance)		35 000
Beam D.L. Stresses(MPa @ 0.5 pt)	F_t	7.42
	F_b	-7.25
Compression Zone – DC + DW (MPa)	F_t	17.37
Compression Zone – 0.5 (DC + DW) + LL +IM (MPa)	F_t	14.43
Compression Zone – DC + DW + LL + IM (MPa)	F_t	23.11
Tension Zone – DC + DW + 0.8 (LL + IM) (MPa)	F_b	-26.10
Factored Moment @ Section (kN-m)	$*M_u$	11 860

$*M_u$ = 1.0 [1.25 DC + 1.50 DW + 1.75 (LL + IM)]

Stresses Due to Prestress

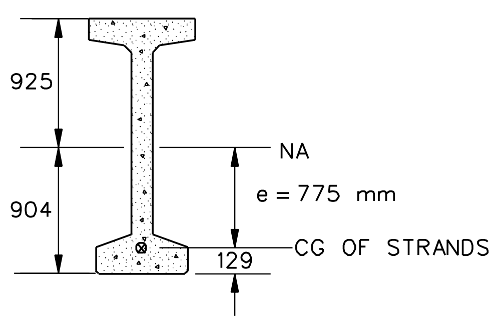
Try 48 strands:

e = 775 mm

A_s = 98.77 mm^2

f_{pu} = 1860 MPa

Estimate losses at transfer to be 8%.



$$P_t = 48(98.77\ \text{mm}^2)\ (0.92)(0.75)(1860\ \text{MPa}) = 6\ 084\ 550\ \text{N}$$

$$\sigma_t = \frac{P}{A} \pm \frac{P_e y}{I}$$

$$\sigma_{t_{top}} = \frac{6\ 084\ 590\ \text{N}}{507\ 000\ \text{mm}^2} - \frac{(6\ 084\ 590\ \text{N})\ (775\ \text{mm})\ (925\ \text{mm})}{2.2806\ \text{x}\ 10^{11}\ \text{mm}^4} = -7.1\ \text{MPa}$$

$$\sigma_{t_{bot}} = \frac{6\ 084\ 590\ \text{N}}{507\ 000\ \text{mm}^2} + \frac{(6\ 084\ 590\ \text{N})\ (775\ \text{mm})\ (904\ \text{mm})}{2.2806\ \text{x}\ 10^{11}\ \text{mm}^4} = 30.7\ \text{MPa}$$

Use Approximate Lump-Sum Estimate at Time-Dependent Losses (LRFD Table 5.9.5.3-1):

$$230\left[1-0.15\left(\frac{f'_c-41}{41}\right)\right]+41\ \text{PPR}$$

$$= 230\left[1-0.15\left(\frac{48-41}{41}\right)\right]+41(1)=265$$

For low-relaxation strands, value may be reduced by 41 MPa:

P_f = 48 (98.77 mm^2) [(0.92)(0.75)(1860) – (265 – 41)] = 5 022 600 N

$$\sigma_{f_{top}} = \frac{5\ 022\ 600\ \text{N}}{507\ 000\ \text{mm}^2} - \frac{(5\ 022\ 600\ \text{N})\ (775\ \text{mm})\ (925\ \text{mm})}{2.2806\ \text{x}\ 10^{11}\ \text{mm}^4} = -5.9\ \text{MPa}$$

$$\sigma_{f_{bot}} = \frac{5\ 022\ 600\ \text{N}}{507\ 000\ \text{mm}^2} + \frac{(5\ 022\ 600\ \text{N})\ (775\ \text{mm})\ (904\ \text{mm})}{2.2806\ \text{x}\ 10^{11}\ \text{mm}^4} = 25.3\ \text{MPa}$$

<u>Check Stresses at Transfer</u>

Compression Stresses (LRFD Article 5.9.4.1.1):

Service I Load Combination: $f_{allow} = 0.60\ f'_{ci}$ = 0.6 (41.5) = 24.9 MPa
-7.3 + 30.7 = 23.4 ≤ 24.9 <u>OK</u>

Tension Stresses (LRFD 5.9.4.1.2):

Service III Load Combination: $f_{allow} = 0.25\ \sqrt{f'_{ci}}\ \leq 1.38$ MPa
= 1.61; therefore, 1.38 MPa controls

7.4 + (–7.1) = 0.3 ≥ –1.38 MPa <u>OK</u>

Check Stresses at Final

Compression Stresses (LRFD Article 5.9.4.2.1):

Service I Load Combination: $f_{allow} = 0.45\ f'_c = 0.45\ (48) = 21.6$ MPa

$17.4 - 5.9 = 11.5 \leq 21.6$ OK

$f_{allow} = 0.40\ f'_c = 19.2$ MPa

$$14.4 - \frac{5.9}{2} = 11.5 \leq 19.2 \quad \text{OK}$$

$f_{allow} = 0.60\ f'_c = 28.8$ MPa

$23.1 - 5.9 = 17.2 \leq 28.8$ OK

Tension Stresses (LRFD Article 5.9.4.2.2):

Service III Load Combination: $f_{allow} = 0.50\sqrt{f'_c} = 3.5$ MPa

$-26.1 + 25.3 = -0.8 \geq -3.5$ OK

Strength I Limit State:

$$\sum \eta_i \gamma_i Q_i = 1.25\ (1830 + 1910 + 55 + 318 + 215) + 1.5\ (183) + 1.75\ (3528) \quad \text{(LRFD Table 3.4.1-1)}$$
$$= 11\,859\ \text{kN} \bullet \text{m}$$

Nominal Flexural Resistance: (LRFD Article 5.7.3)

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \quad \text{(LRFD Equation 5.7.3.1.1-1)}$$

$$d_p = (1829 + 20 + 165) - 129$$
$$= 1885\ \text{mm}$$

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \quad \text{(LRFD Equation 5.7.3.1.1-2)}$$
$$= 2\left(1.04 - \frac{1675}{1860}\right)$$
$$= 0.279$$

$$\beta_1 = 0.85 - 0.5\left(\frac{31 - 28}{7}\right) = 0.83 \quad \text{(LRFD Article 5.7.2)}$$

Assume rectangular section behavior:

$$c = \frac{A_{ps}\ f_{pu}}{0.85\ f'_c\ \beta_1 b + KA_{ps}\ \dfrac{f_{pu}}{d_p}} \qquad \text{(LRFD Equation 5.7.3.1.1-4)}$$

$$= \frac{(48)\,(98.77)\,(1860)}{0.85\,(31)\,(0.83)(2361) + (0.279)\,(48)\,(98.77)\ \dfrac{1860}{1885}}$$

= 166.6 mm (just greater than 165 mm; say ok)

$$f_{ps} = 1860\left(1 - 0.279\,\frac{166.6}{1885}\right)$$

= 1814 MPa

$a = \beta_1 c = (0.83)\ (166.6) = 138$ mm (LRFD Article 5.7.2.2)

$$M_n = A_{ps}\ f_{ps}\left(d_p - \frac{a}{2}\right) \qquad \text{(LRFD Equation 5.7.3.2.2-1)}$$

$$= 48\,(98.77\ \text{mm}^2)\,(1814\ \text{MPa})\left[1885\ \text{mm} - \frac{138\ \text{mm}}{2}\right]$$

As is typical, the design is governed by the service limit states, not the strength limit states.

$$= 15\,618\ \ \text{kN} \bullet \text{m}\ (> 11\ \ 860\ \text{kN} \bullet \text{m})\ (\text{OK})$$

17.6 GIRDER DETAILS

17.6.1 Fabrication Lengths

The overall length of fabricated girders shall be increased 0.6 mm per m of length to compensate for elastic shortening, shrinkage and creep.

The length of fabricated girders shall also be adjusted for highway grades.

17.6.2 Camber and Dead-Load Deflection

Dead-load deflections shall be tabulated on erection plan drawings. A typical dead-load deflection table is shown in Figure 17.6A.

A note appears on Standard Drawings MSL-5 and MSL-6 directing the contractor's attention to the fact that the camber of the individual prestressed girders may vary. The contractor shall account for this variation in camber, determined in the field at 0.1 points, by varying the haunch depth as required.

17.6.3 Girder End Details

Typical girder end details, shear reinforcement and confinement steel are shown for the standard girders on the Standard Drawings MB-1, MB-A, MB-4, MM-72 and MMT-28. Modified girder end details will be required on the plans if the bridge is constructed on a skew.

17.6.4 Bearing Shoes

Fixed bearing shoes shown on Standard Drawings MB-1, MB-A, MB-4, MM-72 and MMT-28 shall be utilized respectively unless otherwise directed. If expansion shoes are required, they are designed specifically to the site conditions and appropriate details are shown on the plans.

17.6.5 Sole Plates

For highway grades greater than or equal to 2%, sole plates shall be beveled to allow for the typically level girder seats.

DEAD LOAD DEFLECTION TABLE					
TYPE 'A' PRESTRESSED CONCRETE BEAM (mm)					
Span Length	Tenth Point				
	0.1	0.2	0.3	0.4	0.5
16 000	4	7	9	11	12

Note: Deflections are symmetrical about the 0.5 point and do not include beam dead load.

TYPICAL DEAD-LOAD DEFLECTION TABLE

Figure 17.6A