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Chapter Sixteen
REINFORCED CONCRETE

Section 5 of the LRFD Bridge Design Specifications provides unified design requirements for concrete in all structural elements — reinforced, prestressed and combinations thereof. Chapter Sixteen presents MDT supplementary information specifically on the general properties of concrete and mild steel reinforcement and the design of reinforced concrete flat slabs. Chapter Seventeen discusses prestressed concrete structures.

16.1 GENERAL

16.1.1 Materials

Reference: LRFD Article 5.4.2

Class SD concrete is specified in two different compressive strengths. For the Missoula and Butte Districts, use 31 MPa. For the Billings, Great Falls and Glendive Districts, use 28 MPa unless there is documentation for the file that concrete strengths were discussed at the Design Parameters Meeting with concurrence from District and Construction staff that 31 MPa concrete is available at the project site.

Figure 16.1A presents MDT's design material properties of concrete for structural applications.

16.1.2 Strut-and-Tie Model

Reference: LRFD Article 5.6.3

Cracked reinforced concrete, reinforced with mild steel reinforcement, prestressing tendons or a combination thereof, ultimately resists load through truss-like load paths. Because reinforced concrete cracks, the compressive-stress trajectories, or struts, within the concrete tend toward straight lines. These compressive-stress trajectories plus the provided steel tensile reinforcement, or ties, form trusses. Because the concrete must be cracked, the strut-and-tie model is not applicable to the service limit states, only the strength and extreme-event limit states. A thorough presentation of the model can be found in Schlaich, J. "Towards a Consistent Design of Structural Concrete," PCI Journal, Vol. 32, No. 3, 1987.

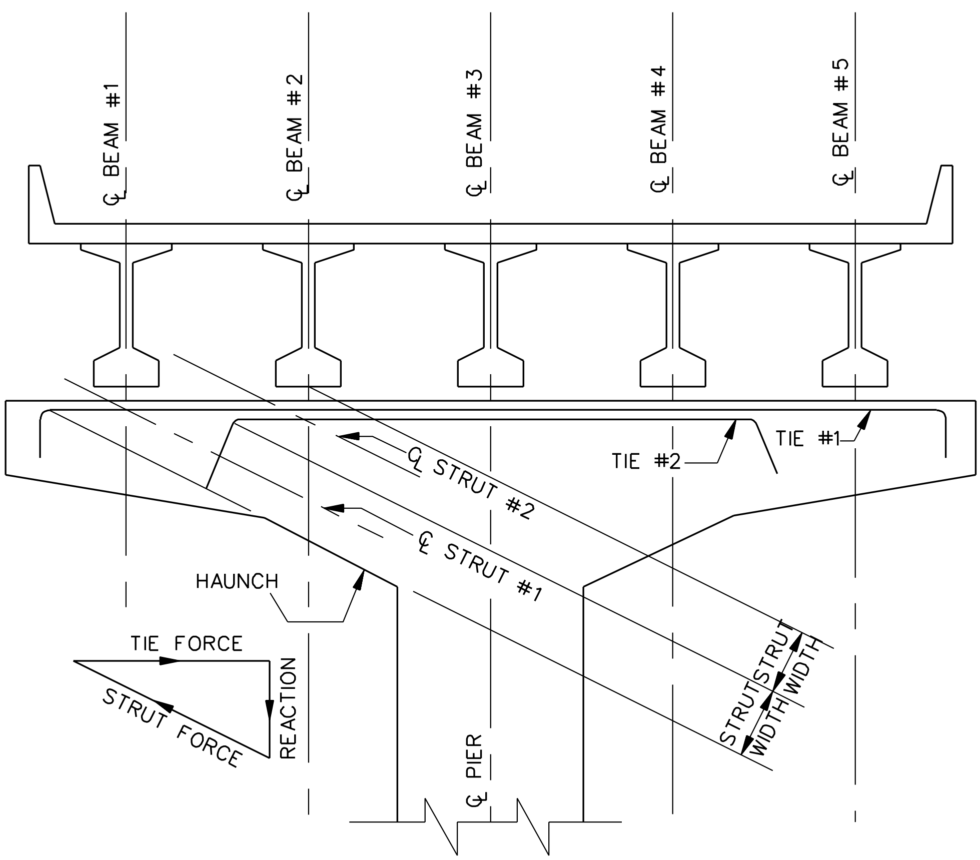
The application of the strut-and-tie model must be approved by the Bridge Area Engineer. Although the model is not typically used for actual proportioning in Montana, it can provide a fast and simple hand check.

Application of the strut-and-tie model for a hammerhead pier is demonstrated in Figure 16.1B. There are five beams supported by the

Concrete	28-day Compressive Strength (f'_c) (MPa)	Modulus of Elasticity (E_c) (MPa)	Modulus of Rupture (f_r) (MPa)
Class DD	21	22 000	2.89
Class Drilled Shaft	21	22 000	2.89
Class SD	28	25 400	3.33
Class SD	31	26 700	3.51

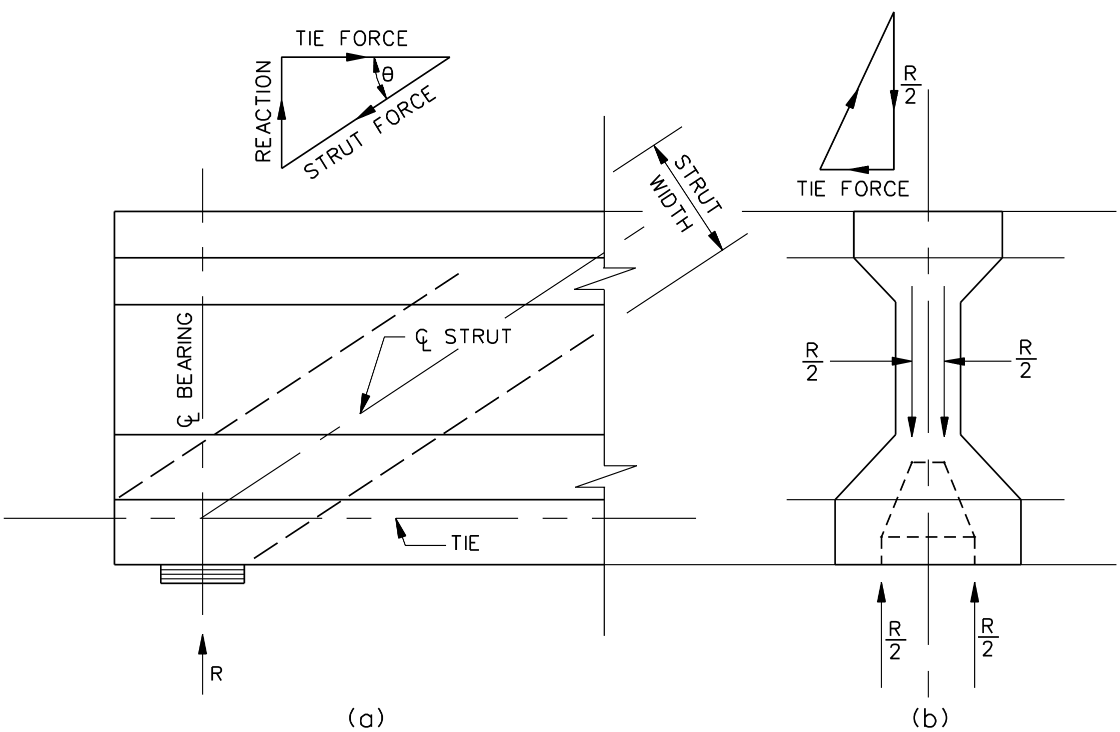
MATERIAL PROPERTIES OF CONCRETE

Figure 16.1A



STRUT-AND-TIE MODEL FOR HAMMERHEAD

Figure 16.1B



STRUT-AND-TIE MODEL FOR BEAM ENDS

Figure 16.1C

pier, of which two affect the design of a cantilever. There are several acceptable truss geometries; the one selected here ensures that the struts, being parallel, are independent from each other. The scheme is indicative of the significance of a well-proportioned cantilever. This design will yield approximately the same amount of steel in both ties. The steel in both ties is extended to the boundaries of their respective struts, then hooked down. The 90° hook of Tie #1 is further secured to the concrete by secondary steel, and the hook of Tie #2 is positioned in, and normal to, Strut #1.

This example was selected because of the potential for excessive cracking of pier heads designed as beams. Normal beam design can be unconservative for this application.

The strut-and-tie model can also be used for the approximate analysis of beam ends. Figure 16.1C(a) shows a convenient way of checking the adequacy of reinforcement in the end-zone and the magnitude of compressive stresses in the web. In lieu of refined calculations, the angle θ may be assumed as 30°. The model illustrates the futility of placing too much vertical (shear) steel in the end zone which is, except for the tie and the strut areas, largely inactive.

Figure 16.1C(b) illustrates an application of the model to estimate the transverse forces in the bearing area to be resisted by the reinforcing cage.

16.1.3 Flexural Resistance

Reference: LRFD Article 5.7

The general flexural-resistance equation of the LRFD Specifications (LRFD Equation 5.7.3.2.2-1) for concrete sections is rewritten below for rectangular concrete sections reinforced with mild steel reinforcement only. The nominal flexural resistance of a rectangular, singly reinforced concrete section is given as:

$$M_n = A_s f_y \left[d_s - 0.5a\right]$$

16.1.4 Limits for Steel Reinforcement

16.1.4.1 Maximum Reinforcement

Reference: LRFD Article 5.7.3.3.1

The LRFD Specifications unifies the maximum allowable steel reinforcement for both reinforced and prestressed sections by limiting the c/d_e ratio to 0.42. This value is based upon the same principles as the traditional limitation of 0.75 ρ_b but is applicable to all concrete sections no matter how reinforced.

16.1.4.2 Minimum Reinforcement

Reference: LRFD Article 5.7.3.3.2

The minimum flexural reinforcement on both faces of a component should provide flexural strength at least equal to the lesser of:

1. 1.2 times the cracking moment of the concrete section assuming the tensile strength as $0.63\sqrt{f'_c}$, or
2. 1.33 times the factored moment required by the governing load combination.

For a rectangular section, since:

$$\sigma = Mc/I = M/S$$

then:

$$M = \sigma S = \sigma bh^2/6$$

and the cracking moment, M_{cr}, is:

$$M_{cr} = 0.63\sqrt{f'_c}\ bh^2/6$$

$$M_{cr} = 0.105\ bh^2\sqrt{f'_c}$$

The factored resistance is:

$M_r = 0.9\ M_n$

but:

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

then:

$$a = \frac{A_s f}{0.85\, f'_c\, b}$$

therefore:

$$M_n = A_s f_y \left(d - \frac{A_s f_y}{2(0.85) f'_c\, b} \right)$$

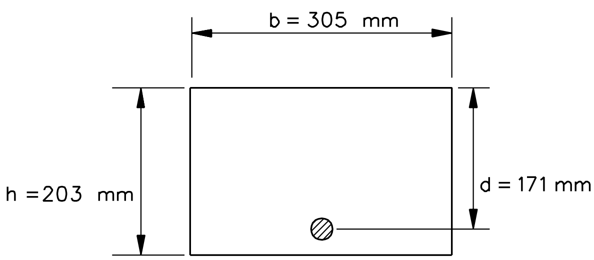
Now, the nominal flexural resistance is no longer a function of the unknown "a." Accordingly:

$$1.2 M_{cr} = 0.9 A_s f_y\, d \left[1.0 - \frac{A_s\, f_y}{1.7 bd\, f'_c} \right]$$

* * * * * * * * * *

Example 16.1.1

See Figure 16.1D for bridge section dimensions.



BRIDGE SECTION DIMENSIONS

Figure 16.1D

b = 305 mm, h = 203 mm and f'_c = 28 MPa:

$$M_{cr} = (0.105)(305)(203^2)\sqrt{28}$$
$$= 6983 \text{ kN - mm}$$

and d = 171 mm and f_y = 420 MPa:

$$B = \frac{(-1.7)(bd\, f'_c)}{f_y}$$
$$= \frac{(-1.7)(305)(171)(28)}{420}$$
$$= (-6937)\, \text{mm}^2$$

$$C = \frac{2.27\, M_{cr} b f'_c}{f_y^2}$$
$$= \frac{(2.533)(6983 \text{ x } 10^3)(305)(28)}{(420)^2}$$
$$= 944 \text{ x } 10^3 \text{ mm}^4$$

from which:

$$A_S = 0.5\left[-B - \sqrt{B^2 - 4C}\right] = 133 \text{ mm}^2$$

or a ratio of ρ = 133 ÷ (305 x 203) = 0.002 15

This process also provides the minimum steel in both directions at top and bottom of concrete flat-slab bridges.

16.1.4.3 Control of Cracking by Distribution of Reinforcement

Reference: LRFD Article 5.7.3.4

These provisions apply to all reinforced concrete flexural members except for bridge deck slabs designed in accordance with the LRFD requirements for empirical decks.

At the Service Limit State, verify that $Z/(d_c A)^{1/3} \leq 0.6 f_y$ (LRFD Equation 5.7.3.4-1).

MDT has typically Severe Exposure (Z = 23 000 N/mm) for bridge deck and barriers and top of hammerhead piers below expansion joints.

For all other conditions, use Moderate Exposure (Z = 30 000 N/mm).

* * * * * * * * * *

16.1.5 Shear Resistance

Reference: LRFD Article 5.8

The LRFD Specifications maintains the traditional sectional approach to shear design in which the nominal shear resistance of a reinforced concrete section is the arithmetic sum of the yield strength of the vertical steel intercepted by the critical crack and the shear resistance of the concrete. The introduction of cot θ in the equation for the steel contribution:

$$V_s = \frac{A_s f_y\ d}{S} \cot\theta$$

signifies that the angle of inclination of diagonal compressive stresses θ could be different from the traditional 45°. In the concrete equation:

$$V_c = 0.083\,\beta\,\sqrt{f'_c}\ bd \times 10^{-3}\ kN$$

The factor "β" determines what multiple of $\sqrt{f'_c}$ may be used as the shear strength of concrete. Both "θ" and "β" are functions of the longitudinal steel strain "ε_x" which, in turn, is a function of "θ." Therefore, the design process is an iterative one. This process may be considered an improvement in accounting for the interaction between shear and flexure and trying to control cracking at strength limit state. It would appear, however, that the amount of vertical steel provided by this process is not substantially different from that given by the traditional approach. Both approaches generally yield conservative results. However, the traditional approach can be seriously unconservative for large members not containing transverse reinforcement. In typical practice, deck slabs and footings are the more common components designed without transverse reinforcement. Typically proportioned slabs and footings are not considered as large members.

The LRFD Specifications provide a simplified procedure for nonprestressed sections in LRFD Article 5.8.3.4.1 that is essentially identical to the traditional approach. This simplified procedure, wherein β and θ are assumed to be 2.0 and 45°, respectively, is the preferred procedure for reinforced concrete sections in Montana.

This simplification results in the following modifications to the equations of LRFD Article 5.8.3.3:

$$V_c = 0.166\sqrt{f'_c}\ b_v d_v$$

$$V_s = \frac{A_v f_y d_v}{S}$$

where the terms are defined in LRFD Article 5.8.3.3.

16.1.6 Fatigue Limit State

Reference: LRFD Article 5.5.3

The fatigue limit state is not normally a critical issue for reinforced concrete structures. Fatigue need not be considered for deck slabs on multiple girders or where the permanent stress f_{min} is compressive and exceeds twice the applied tensile live-load stress due to the fatigue load combination.

Assuming r/h = 0.3, the allowable stress range of LRFD Equation 5.5.3.2-1 may be rearranged for easier interpretations:

$$f_f + 0.33\ f_{min} \leq 161\ MPa$$

The LRFD Specifications presents a major change in computing the applied stress range. It is the stress range due to 75% of a single truck per bridge (lane load excluded) with reduced impact and with the major axles of the truck at a constant spacing of 9 m, instead of all contributing lanes being loaded. Also, the LRFD Specifications specifies that, when the bridge is analyzed by the approximate distribution method, live-load distribution factors for one design lane loaded shall be used.

16.2 STEEL REINFORCEMENT

16.2.1 General

Reference: LRFD Article 5.4.3.1

Steel reinforcement shall consist of either uncoated, "black" rebars or epoxy-coated rebars according to these Specifications. Generally, reinforcing bars should conform to the requirements of ASTM A615/A615M, Grade 420 with a 420 MPa yield strength. For seismic applications, rebars conforming to ASTM A706 should be specified for greater quality control of unanticipated overstrength.

16.2.2 Sizes

Reinforcing bars are referred to in the contract plans and specifications by number, and they vary in size from #13 to #57. Figure 16.2A shows the sizes and various properties of the typical bars used in Montana.

To avoid handling damage, the minimum bar size shall be #13.

16.2.3 Lengths

To facilitate handling, the maximum length of #13 reinforcing bars is 12.19 m. Larger bars can be specified in lengths up to 18.29 m.

16.2.4 Concrete Cover

Reference: LRFD Article 5.12.3

See Figure 16.2B for MDT criteria for minimum concrete cover for various applications. All clearances to reinforcing steel shall be shown on the plans.

16.2.5 Spacing of Reinforcement

Reference: LRFD Article 5.10.3

For minimum spacing of bars, see Figure 16.2C. Fit and clearance of reinforcing shall be carefully checked by calculations and large-scale drawings. Skews tend to aggravate problems of reinforcing interference. Tolerances normally allowed for cutting, bending and locating reinforcing shall be considered.

Common areas of interference are:

Bar Size Designation	Nominal Dimensions		
	Mass (kg/m)	Diameter (mm)	Area (mm^2)
#13	0.994	12.7	129
#16	1.552	15.9	199
#19	2.235	19.1	284
#22	3.042	22.2	387
#25	3.973	25.4	510
#29	5.060	28.7	645
#32	6.404	32.3	819
#36	7.907	35.8	1006
#43	11.38	43.0	1452
#57	20.24	57.3	2581

REINFORCEMENT BAR PROPERTIES

Figure 16.2A

Item	Minimum Concrete Cover for Design & Detailing (mm)
Deck Slabs (including both slabs on girders and flat-slab bridges) Top Steel Bottom Steel	 60 25
Footings and Pier Shafts	50
Stirrups and Ties	40
Items cast against ground	80
Drilled Shaft	75
All Other Structural Elements	50

CONCRETE COVER

Figure 16.2B

Bar Size	Preferred Minimum Spacing (mm)	
	Unspliced Bars	Spliced Bars
#13	50	65
#16	54	70
#19	58	78
#22	60	82
#25	64	90
#29	72	100
#32	80	112
#36	90	N/A
#43	108	N/A
#57	143	N/A

Note: Minimum spacing values are based upon Articles 5.10.3.1.1 and 5.10.3.1.4 in the LRFD Specifications and the nominal diameters of metric reinforcing bars. In Montana, the maximum size of coarse aggregate used in both cast-in-place and precast concrete is 19 mm.

MINIMUM SPACING OF BARS

Figure 16.2C

1. between slab reinforcing and reinforcing in monolithic end bents or intermediate bents;

2. vertical column bars projecting through main reinforcing in pier caps;

3. the areas near expansion devices;

4. anchor plates for steel girders; and

5. between prestressing steel and reinforcing steel stirrups, ties, etc.

Show the clear distance from the face of concrete to the first bar. When the distance between the first and last bars is such that the number of bars required results in spacings in increments of other than 5 mm, show the bars to be equally spaced.

16.2.6 Development of Reinforcement

Reference: LRFD Article 5.11.2

Reinforcement is required to be developed on both sides of a point of maximum stress at any section of a reinforced concrete member. This requirement is specified in terms of a development length, l_d.

16.2.6.1 Development Length in Tension

Development of bars in tension involves calculating the basic development length, l_{db}. The development length is modified by factors to reflect bar spacing, cover, enclosing transverse reinforcement, top bar effect, type of aggregate, epoxy coating and the ratio of the required area to the provided area of reinforcement.

The development length, l_d (including all applicable modification factors) must not be less than 300 mm.

Figures 16.2D through 16.2G show the tension development length for both uncoated and epoxy-coated Grade 420 bars for normal weight concrete with specified strengths of 21 and 28 MPa. For Class SD concrete with f'_c = 31 MPa, use development lengths shown for f'_c = 28 MPa.

16.2.6.2 Development Length in Compression

The standard procedure in Montana is to use tension development lengths for bars in either tension or in compression. This ensures that an adequate development length will be provided in a compression member that may be primarily controlled by bending. Hooks are not considered effective in developing bars in compression. When designing column bars with hooks to develop the tension, ensure that the straight length is also an adequate length to develop the bar in compression.

16.2.6.3 Standard End Hook Development Length in Tension

Standard end hooks, utilizing 90- and 180-degree end hooks, are used to develop bars in tension where space limitations restrict the use of straight bars. End hooks on compression bars are not effective for development length purposes. The values shown in Figures 16.2H through 16.2J show the tension development lengths for normal weight concrete with specified strengths of 21 and 28 MPa. For Class SD concrete with f'_c = 31 MPa, use development lengths shown for f'_c = 28 MPa.

Figure 16.2K illustrates the hooked-bar details for the development of standard hooks.

Figure 16.2D
Tension Development Lengths(l_d) for Grade 420
Uncoated Bars; f'_c = 21 MPa; Normal Weight Concrete

Bar Size	l_d		l_d Modified for Bar Spacing Per Article 5.11.2.1.3 (Spacing ≥ 150 mm)	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	450	320	360	260
#16	570	410	450	330
#19	730	530	590	420
#22	1000	710	800	570
#25	1310	940	1050	750
#29	1660	1190	1330	950
#32	2110	1510	1690	1210
#36	2590	1850	2070	1480
#43	3210	2300	2570	1840
#57	4370	3120	3490	2500

Figure 16.2E
Tension Development Lengths (l_d) for Grade 420
Uncoated Bars; f'_c = 28 MPa; Normal Weight Concrete

Bar Size	l_d		l_d Modified for Bar Spacing Per Article 5.11.2.1.3 (Spacing ≥ 150 mm)	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	450	320	360	260
#16	570	410	450	330
#19	680	490	540	390
#22	860	620	690	500
#25	1140	810	910	650
#29	1440	1030	1150	820
#32	1830	1310	1460	1050
#36	2240	1600	1790	1280
#43	2780	1990	2230	1590
#57	3780	2700	3030	2160

STRAIGHT UNCOATED DEFORMED BARS

Figure 16.2F
Tension Development Lengths (l_d) for Grade 420 Epoxy
Coated Bars; f'_c = 21 MPa; Normal Weight Concrete

Bar Size	l_d		l_d Modified for Bar Spacing Per Article 5.11.2.1.3 (Spacing ≥ 150 mm)	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	540	390	430	310
#16	680	490	540	390
#19	880	630	700	500
#22	1200	860	960	690
#25	1580	1130	1260	900
#29	1990	1420	1590	1140
#32	2530	1810	2200	1450
#36	3100	2220	2480	1770
#43	3850	2750	3080	2200
#57	5240	3740	4190	3000

Figure 16.2G
Tension Development Lengths (l_d) for Grade 420 Epoxy
Coated Bars; f'_c = 28 MPa; Normal Weight Concrete

Bar Size	l_d		l_d Modified for Bar Spacing Per Article 5.11.2.1.3 (Spacing ≥ 150 mm)	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	540	390	430	310
#16	680	490	540	390
#19	810	580	650	470
#22	1040	740	830	590
#25	1370	980	1090	780
#29	1720	1230	1340	990
#32	2190	1570	1750	1250
#36	2690	1920	2150	1540
#43	3340	2390	2670	1910
#57	4540	3240	3630	2600

STRAIGHT EPOXY COATED DEFORMED BARS

Note: The shaded cells indicate where the tabularized tension development lengths do not meet the compressive development length requirements of LRFD Article 5.11.2.2.1.

Figure 16.2H
Tension Development Lengths(l_{dh}) Grade 420
Uncoated Bars; f'_c = 21 MPa; Normal Weight Concrete

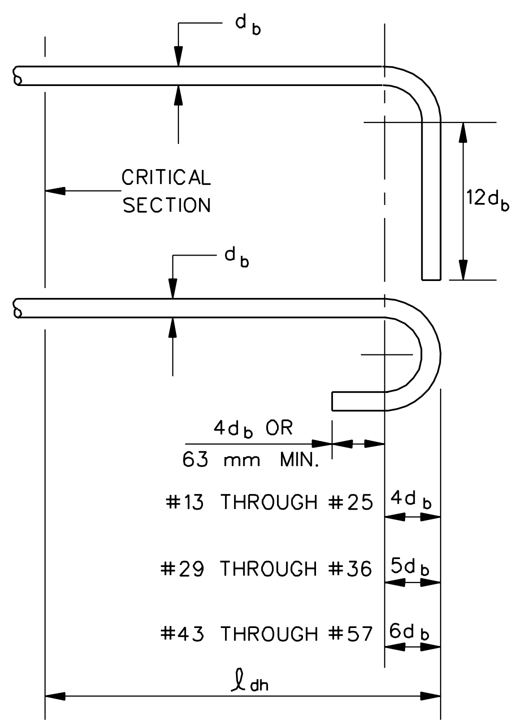
Bar Size	l_{dh} Side Cover < 60 mm or Cover on Tail <50 mm $l_{dh} = l_{db}$ (mm)	l_{dh} Side Cover ≥ 60 mm and Cover on Tail ≥ 50 mm $l_{dh} = 0.7\ l_{db}$ (mm)
#13	280	200
#16	350	250
#19	420	300
#22	490	350
#25	560	400
#29	630	450
#32	710	500
#36	790	560
#43	950	950
#57	1260	1260

Figure 16.2I
Tension Development Lengths (l_{dh}) Grade 420
Uncoated Bars; f'_c = 28 MPa; Normal Weight Concrete

Bar Size	l_{dh} Side Cover < 60 mm or Cover on Tail <50 mm $l_{dh} = l_{db}$ (mm)	l_{dh} Side Cover ≥ 60 mm and Cover on Tail ≥ 50 mm $l_{dh} = 0.7\ l_{db}$ (mm)
#13	250	180
#16	310	220
#19	370	260
#22	430	310
#25	490	350
#29	550	390
#32	620	440
#36	690	490
#43	820	820
#57	1090	1090

Figure 16.2J
Hook Lengths After Bends

Bar Size	Hook Length After 90° Bend (mm)	Hook Length After 180° Bend (mm)
#13	160	70
#16	200	70
#19	230	80
#22	270	90
#25	310	110
#29	350	120
#32	390	130
#36	430	150
#43	520	180
#57	690	230



Hooked-Bar Details for Development of Standard Hooks
Figure 16.2K

Note: *Development lengths shown to be multiplied by a factor of 1.2 for epoxy coated bars.*

16.2.7 Splices

Reference: LRFD Article 5.11.5

16.2.7.1 General

Three methods may be used to splice reinforcing bars — lap splices, mechanical splices and welded splices. Lap splicing of reinforcing bars is the most common method. Lap splices are not allowed in potential plastic hinge regions. To minimize the possibility of mislocated lap splices, the plans should clearly show the locations and lengths of all lap splices. Due to splice lengths required, lap splices are not permitted for bars larger than #36; if bars larger than #36 are necessary, mechanical bar splices shall be used.

No lap splices, for either tension or compression bars, shall be less than 310 mm.

If transverse reinforcing steel in a bridge deck will be lapped near a longitudinal construction joint, show the entire lap splice on the side of the construction joint that will be poured last.

16.2.7.2 Lap Splices — Tension

Reference: LRFD Article 5.11.5.3

Many of the same factors which affect development length affect splices. Consequently, tension lap splices are a function of the bar development length (l_d). All lap splices in tension should be detailed as Class C tension lap splices unless problems arise. The other lesser classes of LRFD Article 5.11.5.3.1 may be used only if the requirements of Class C cannot be satisfied. Designers are encouraged to splice bars at points of minimum stress.

For tension splices, the length of a lap splice between bars of different sizes shall be governed by the smaller bar.

Figures 16.2L through 16.2O show tension lap splices for both uncoated and epoxy coated Grade 420 bars for normal weight concrete with specified strengths of 21 and 28 MPa. For Class SD concrete with specified strength of 31 MPa, use the tabularized values for 28 MPa.

16.2.7.3 Lap Splices — Compression

In Montana, lap splices in compression members are sized for tension lap splices. The design of compression members, such as columns, pier walls and abutment walls, involves the combination of vertical and lateral loads. The policy of requiring a tension lap splice considers the possibility that the member may be primarily controlled by bending. The increase in cost of additional splice reinforcement material is minimal.

16.2.7.4 Mechanical Splices

A second method of splicing is by mechanical splices, which are proprietary splicing mechanisms. Mechanical splices are appropriate where interference problems preclude the use of more conventional lap splices and in phased construction. Even with mechanical splices, it is frequently necessary to stagger splices. The designer must check clearances. The requirements for mechanical splices are found in Articles 5.11.5.2.2, 5.11.5.3.2 and 5.11.5.5.2 of the LRFD Specifications. Epoxy-coated mechanical splices must be used with epoxy-coated reinforcing steel.

16.2.7.5 Welded Splices

Splicing of reinforcing bars by welding, although allowed by the LRFD Specifications, is seldom used by MDT and not encouraged principally because of quality issues with field welding. However, it is common practice to weld the tail of a column spiral reinforcing back on itself. According to the LRFD Specifications within plastic hinge zones, reinforcing steel splices are limited to welded splices or mechanical connectors. Those provisions of the Specifications may make welded splices the best choice in certain circumstances. Welding reinforcing steel is not covered by the **AASHTO/ANSI/AWS D1.5 Bridge Welding Code**, and the current **Structural Welding Code — Reinforcing Steel** of AWS (D1.4) must be referenced. The AASHTO Code does not allow welded splices in decks.

All reinforcing steel used by MDT can be welded; however, if reinforcing steel is to be welded, A-706M reinforcing steel is preferred due to tighter controls on the carbon content. The carbon content determines preheat requirements for welding.

16.2.8 Epoxy-Coated Reinforcement

Reference: LRFD Articles 2.5.2.1.1 and 5.12.4

MDT uses epoxy-coated reinforcement at the following locations:

1. all bridge deck reinforcement;

2. all reinforcing that extend into the slabs, including cast-in-place concrete diaphragm shear steel and excluding prestressed girder shear connectors;

3. vertical back wall and back wall connection steel extending into the slab for structures located on the State highway system (not for off-system bridges);

4. cap shear and primary reinforcement of caps located under deck expansion joints. Also include beam seat and shear block reinforcement at these locations; and

5. all reinforcing in bridge approach slabs.

For other locations, use plain reinforcing steel. For example:

1. bridge deck and deck joint rehabilitations of existing bridges with plain steel;

2. all substructure reinforcement including footings, piers, columns and caps not specifically identified above as needing epoxy bars;

3. pile reinforcing at pipe-pile-to-cap connections, except for integral caps where the vertical bars extend into the limits of the slab;

4. wing wall reinforcement for typical and turnback wings; and

5. reinforcing for typical reinforced concrete retaining walls.

16.2.9 Detailing of Reinforcement

16.2.9.1 Spirals

Figure 16.2P illustrates the detailing of spiral reinforcement. In the Bill of Reinforcing both the height of the spiral and the length of the bent bar should be indicated plus the pitch spacing and spiral radius.

Figure 16.2L
Tension Lap Splice Lengths for Grade 420 Uncoated Bars; f'_c = 21 MPa; Normal Weight Concrete

Bar Size	Center-to-Center Spacing < 150 mm or < 75 mm from side face of member		Center-to-Center Spacing ≥ 150 mm and ≥ 75 mm from side face of member	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	750	530	620	430
#16	940	670	750	550
#19	1180	840	960	680
#22	1590	1130	1280	910
#25	2080	1480	1670	1190
#29	2620	1870	2120	1500
#32	3340	2380	2670	1910
#36	4140	2950	3320	2370
#43	Lap Splices Not Allowed			
#57				

Figure 16.2M
Tension Lap Splice Lengths for Grade 420 Uncoated Bars; f'_c = 28 MPa; Normal Weight Concrete

Bar Size	Center-to-Center Spacing < 150 mm or < 75 mm from side face of member		Center-to-Center Spacing ≥ 150 mm and ≥ 75 mm from side face of member	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	750	530	620	430
#16	940	670	750	550
#19	1110	790	890	630
#22	1400	990	1130	800
#25	1790	1280	1430	1020
#29	2300	1640	1840	1310
#32	2890	2060	2320	1650
#36	3560	2540	2860	2040
#43	Lap Splices Not Allowed			
#57				

CLASS C — Uncoated Bars

Figure 16.2N
Tension Lap Splice Lengths for Grade 420 Epoxy Coated Bars; f'_c = 21 MPa; Normal Weight Concrete
(Bar cover ≥ $3d_b$ and clear spacing between bars ≥ $6d_b$)

Bar Size	Center-to-Center Spacing < 150 mm or < 75 mm from side face of member		Center-to-Center Spacing ≥ 150 mm and ≥ 75 mm from side face of member	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	910	650	750	510
#16	1130	800	910	670
#19	1420	1010	1160	820
#22	1910	1360	1530	1090
#25	2500	1790	2010	1430
#29	3150	2250	2540	1810
#32	4020	2860	3220	2300
#36	4970	3540	3980	2840
#43	Lap Splices Not Allowed			
#57				

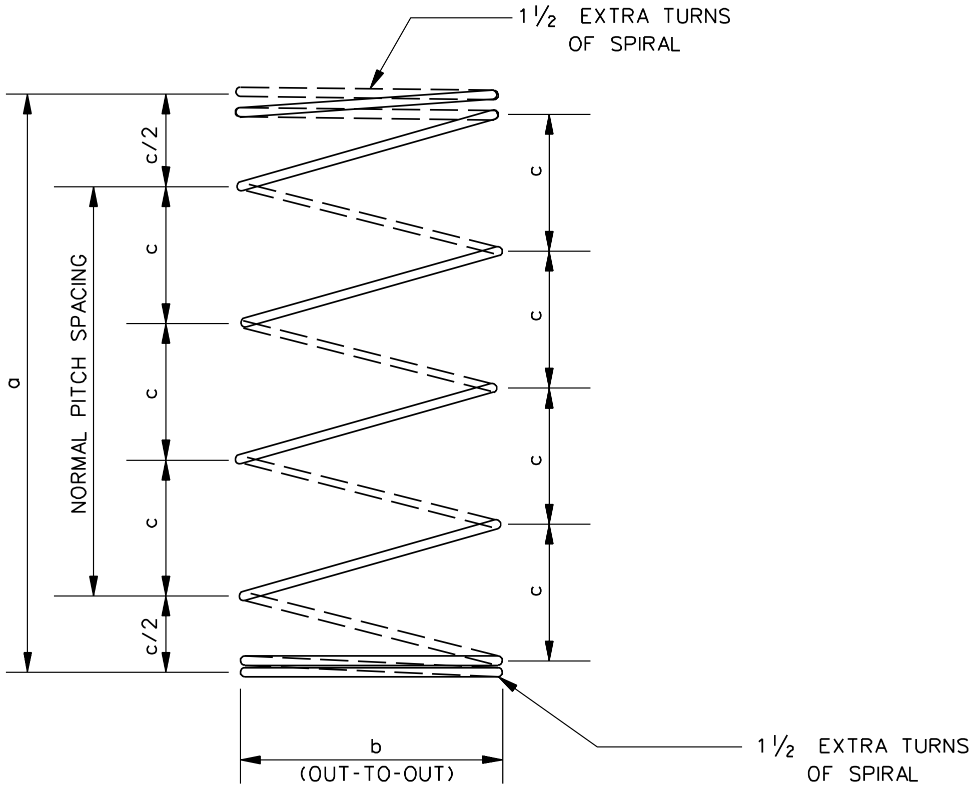
Figure 16.2O
Tension Lap Splice Lengths for Grade 420 Epoxy Coated Bars; f'_c = 28 MPa; Normal Weight Concrete
(Bar cover ≥ $3d_b$ and clear spacing between bars ≥ $6d_b$)

Bar Size	Center-to-Center Spacing < 150 mm or < 75 mm from side face of member		Center-to-Center Spacing ≥ 150 mm and ≥ 75 mm from side face of member	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	910	650	750	510
#16	1130	800	910	670
#19	1330	960	1080	770
#22	1690	1190	1360	970
#25	2150	1530	1720	1230
#29	2760	1980	2210	1590
#32	3470	2490	2790	1990
#36	4270	3050	3440	2450
#43	Lap Splices Not Allowed			
#57				

CLASS C — Epoxy-Coated Bars

Top bars are horizontal bars so placed that more than 300 mm of fresh concrete is cast in the member below the bar.

Splice lengths shown in the Figures for both uncoated and epoxy-coated bars must be multiplied by a factor of 2.0 for bars with a cover of d_b or less, or with a clear spacing between bars of $2d_b$ or less, where d_b equals the bar diameter.



KEY:

a = SPIRAL HEIGHT
b = OUTSIDE DIAMETER
c = PITCH
d = BAR DIAMETER
L = TOTAL LENGTH OF SPIRAL REINFORCEMENT

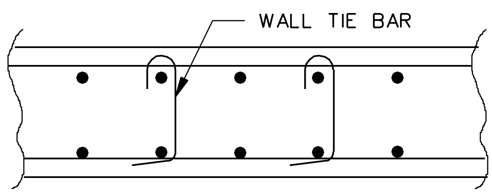
$L = [(a/c + 2(1\frac{1}{2}\text{TURNS})]\pi b$

SPIRAL REINFORCEMENT

Figure 16.2P

16.2.9.2 Wall Tie Bars

Use wall tie bars with a 180° hook on one end and a 90° hook on the other. This deviation from the LRFD Specifications is allowed to facilitate placement of the ties. See Figure 16.2Q for an example of a wall tie bar.



WALL PLAN VIEW
Figure 16.2Q

16.2.9.3 Drilled Shaft Cages

Drilled shaft reinforcing cages have often in the past been detailed with cross ties. The purpose of these ties is to stabilize the cage during placement of the cage into the shaft. Experience has shown that cross ties frequently make concrete placement more difficult. Because the stability of the reinforcing cage is the contractor's responsibility and cross ties are only used during construction, it is not necessary to show them on the plans.

To assure good concrete flow through the reinforcing cage, the clear spacing of all bars should not be less than 5 times the coarse aggregate size. To meet this requirement, bundled bars are allowed. This spacing requirement applies universally, whether in splices, spirals or hoops or among the main reinforcement. Use lap or mechanical splices and place splices at the lower end of the cage to provide lower splice stresses due to service loads and cage handling. Stagger mechanical splices in adjacent bars a minimum of 600 mm. Be aware that using hooks at the top ends of vertical reinforcement can complicate casing extraction and the placement of cap cages. Hook orientation should be determined by the design engineer and shown on the plans.

16.3 REINFORCED CAST-IN-PLACE CONCRETE FLAT SLABS

16.3.1 General

Reference: LRFD Article 5.14.4

The superstructures typically called "flat slabs" in Montana are termed "slab superstructures" in the LRFD Specifications.

The reinforced cast-in-place concrete flat slab bridge is frequently used by MDT because of its suitability for short spans and its ease of construction. It is the simplest among all superstructure systems.

Section 16.3 presents information for the design of reinforced cast-in-place concrete flat slabs that amplify or clarify the provisions in the LRFD Bridge Design Specifications. The Section also presents design information specific to MDT practices.

16.3.1.1 Materials

Reference: LRFD Article 5.4

Use Class SD concrete for reinforced concrete flat slabs. See Figure 16.1A for concrete properties.

16.3.1.2 Cover

Reference: LRFD Article 5.12.3

Figure 16.3A presents MDT criteria for minimum concrete cover for various elements of reinforced concrete flat slabs. All clearances to reinforcing steel shall be shown on the plans.

16.3.1.3 Haunches

In general, MDT prefers straight haunches over parabolic haunches because straight haunches are comparatively easy to form yet result in relatively good stress flow.

Haunching is used to decrease maximum positive moments in continuous structures by attracting more negative moments to the haunches and to provide adequate resistance at the haunches for the increased negative moments. It is a simple, effective and economical way to enhance the resistance of thin concrete flat slabs.

The preferable ratio between the end and intermediate spans is approximately 0.75 to 0.80, but the system permits considerable freedom in selecting span ratios. The ratio between the depths at the edge of intermediate pier cap and at the point of maximum positive moment should be approximately 1.2. Except for aesthetic reasons, the length of the haunch should be approximately 0.15L, where "L" is the intermediate span length; longer haunches may

Element	Minimum Concrete Cover
Top of Slab Bottom of Slab Ends of Slab Edge of Slab	60 mm* 25 mm 40 mm 75 mm

**This includes a 35-mm sacrificial wearing surface.*

MINIMUM CONCRETE COVER
(Reinforced Concrete Flat Slabs)

Figure 16.3A

be unnecessarily expensive and/or structurally counterproductive.

16.3.1.4 Minimum Reinforcement

Reference: LRFD Articles 5.7.3.3.2, 5.10.6 and 5.14.4.1

In both the longitudinal and transverse directions, at both the top and bottom of the slab, the minimum reinforcement should be determined according to the provisions of Articles 5.7.3.3.2 and 5.10.8 in the LRFD Specifications. The first is based on the cracking flexural strength of a component, and the second reflects requirements for shrinkage and temperature. In flat slabs, the two articles provide nearly identical amounts of minimum reinforcement in the majority of cases.

According to Article 5.14.4.1 of the LRFD Specifications, bottom transverse reinforcement, the above-minimum provisions notwithstanding, may be determined either by two-dimensional analysis or as a percentage of the maximum longitudinal positive moment steel in accordance with LRFD Equation 5.14.4.1-1. The span length, L, in the equation should be taken as that measured from the centerline to centerline of the supports. Especially for heavily skewed and/or curved bridges, the analytical approach is recommended.

Section 16.3.4 gives a simplified approach for shrinkage and temperature steel requirements.

16.3.2 Construction Joints

Transverse construction joints are not allowed for design purposes in reinforced concrete flat slabs. However, because of construction problems, they may become unavoidable. The **MDT Standard Specifications** provides construction requirements where transverse construction joints are unavoidable.

Longitudinal construction joints in reinforced concrete slab bridges are also undesirable. However, bridge width, phase construction, the method of placing concrete, rate of delivery of concrete, and the type of finishing machine used by the contractor may dictate whether or not a reinforced concrete slab bridge can be poured in a single pour.

If the slab structure will be built in phases, show the entire lap splice for all transverse reinforcing steel on the side of the construction joint that will be poured last.

16.3.3 Longitudinal Edge Beam Design

Reference: LRFD Articles 5.14.4.1, 9.7.1.4 and 4.6.2.1.4

Edge beams must be provided along the edges of flat slabs. The edge beams can be thickened sections and/or more heavily reinforced sections composite with the slab. The width of the edge beams may be taken to be the width of the equivalent strip used in analysis as described in Section 16.3.8.

16.3.4 Shrinkage and Temperature Reinforcement

Reference: LRFD Article 5.6.2

MDT practice is that evaluating the redistribution of force effects as a result of shrinkage, temperature change, creep and movements of supports is not necessary when designing reinforced concrete flat slabs.

Shrinkage and temperature reinforcement is #13s @ 300 mm.

16.3.5 Reinforcing Steel and Constructibility

The following practices for reinforcing steel should be met to improve the constructibility of reinforced concrete flat slabs:

1. The maximum reinforcing bar size shall be #36.

2. The minimum spacing of reinforcing bars shall be 100 mm.

16.3.6 Deck Drainage

Reference: LRFD Article 2.6.6

Section 15.3.8 discusses drainage for bridge decks in conjunction with prestressed concrete or structural steel superstructures. This information also applies to reinforced concrete flat slabs.

16.3.7 Distribution of Concrete Barrier Railing Dead Load

The edge beam carries the dead load of the barrier.

16.3.8 Distribution of Live Load

Reference: LRFD Articles 4.6.2.3 and 4.6.2.1.4

Section 14.3.2 discusses the application of vehicular live load, and Section 15.2 discusses the application of the Strip Method to bridge decks. The following specifically applies to the distribution of live load to reinforced concrete flat slabs:

1. For continuous flat slabs with variable span lengths, one equivalent strip width (E) shall be developed using the shortest span length for the value of L_1. This strip width should be used for moments throughout the entire length of the bridge.

2. The equivalent strip width (E) is the transverse width of slab over which an "axle" unit is distributed.

3. Different strip widths are specified for the flat slab itself and its edge girders in LRFD Articles 4.6.2.3 and 4.6.2.1.4, respectively.

4. In most cases, using Equation 4.6.2.3-3 from the LRFD Specifications for the reduction of moments in skewed slab-type bridges will not significantly change the reinforcing steel requirements. Therefore, for simplicity of design, the Department does not require the use of the reduction factor "r."

16.3.9 Shear Resistance

Reference: LRFD Article 5.14.4.1

Single-span and continuous-span flat slabs designed for moment in conformance with Article 4.6.2.3 of the LRFD Specifications may be considered satisfactory for shear.

16.3.10 Minimum Thickness of Slab

Reference: LRFD Article 2.5.2.6.3

For the typical MDT three-span continuous flat-slab bridges of total lengths of 18 300 mm and 23 775 mm, the minimum slab thicknesses are 360 mm and 410 mm, respectively. These specified minimum thicknesses include a 35-mm sacrificial wearing surface.

In the event of clearance problems and with the approval of the Crew Chief, the minimum slab thickness requirements in accordance with LRFD Table 2.5.2.6.3-1 may be used. In using the equations in the LRFD Table, it is assumed that:

1. S is the length of the longest span.

2. The calculated thickness includes the 35-mm sacrificial wearing surface.

3. The thickness used may be greater than the minimum if needed for design.

16.3.11 Development of Flexural Reinforcement

Reference: LRFD Article 5.11.1.2

Article 5.11.1.2 of the LRFD Specifications presents specifications for the portion of the longitudinal positive moment reinforcement that must be extended past the point required by the factored maximum moment diagram. Similarly, there is a more stringent provision addressing the location of the anchorage for the longitudinal negative moment reinforcement.

16.3.12 Skews on Reinforced Concrete Slab Bridges

Reference: LRFD Article 9.7.1.3

For up to a 25° skew angle, the transverse reinforcement is permitted to run parallel to the skew, providing for equal bar lengths. In excess of 25°, the transverse reinforcement should be placed perpendicular to the longitudinal reinforcement. This provision concerns the direction of principal tensile stresses as they develop in heavily skewed structures and is intended to prevent excessive cracking.

16.3.13 Substructures

The following describes typical MDT practice for types of substructures used in conjunction with reinforced concrete flat slabs:

1. End Supports. Where possible, use integral or semi-integral abutments. In general, their use is not restricted by highway alignment nor skew; the maximum length is 60 m for use of integral abutments without a special analysis. See Sections 13.4 and 19.1 for more information on end supports, including the use of non-integral abutments and the use of integral abutments where the bridge length exceeds 60 m.

2. Intermediate Supports. See Section 19.2 for typical MDT practices for the selection of the type of intermediate support (e.g., wall piers, pipe pile bents and multiple column bents).

16.3.13.1 Design Details for Integral Caps at Intermediate Bents of Flat Slabs

The following presents specific design details which represent typical MDT practices for the design of integral caps in conjunction with reinforced concrete flat slabs:

1. MDT's standard cap dimensions are 1000 mm in width and a depth of 1000 mm from the top of the slab to the break point in the crown.

2. All shear reinforcement in the caps is placed parallel to the longitudinal slab steel.

3. Standard pile embedment into the abutment and intermediate bent caps is 500 mm.

4. A 10-mm drip groove shall be located 50 mm in from the edge of the slab.

5. The profile of the bottom of the bent cap can be made level if the difference in top-of-slab elevations at the left and right edge of slab, along the centerline of the bent cap, is 75 mm or less. For a difference of greater than 75 mm, slope the bottom of the cap.

Figure 16.3C illustrates a typical section of an integral cap at an intermediate bent of a flat slab.

16.3.13.2 Design Details for End Bents of Flat Slabs

Flat slab end bents typically consist of a pile cap with a backwall constructed above it. The backwall will either be connected to normal straight wingwalls or turnback wingwalls. The backwall height is selected based on burying the bottom of the pile cap in the abutment slope a minimum of 700 mm and allowing 1000 mm of headroom between the abutment slope and the bottom of the slab to aid in inspection.

The slab superstructure is connected to the backwall by a series of 25-mm diameter steel dowels made with metal or PVC expansion caps. This connection creates a hinge condition and allows for future jacking of the slab superstructure in case of settlement. A waterstop is placed between the slab and backwall to control seepage.

For normal crown structures, the pile cap is built level and the backwall height is varied. Typically, one height of U-bar is used for vertical reinforcement, and the embedment into the cap is varied to match the crown of the roadway. On superelevated structures, slope the pile cap to match the superelevation. MDT's minimum cap dimensions are 1000-mm in width and 800 mm in depth. Standard pile embedment into the cap is 500 mm.

Figure 16.3D illustrates a typical section of an end bent with a flat slab.

16.3.14 <u>Sample Design for Flat Slab</u>

16.3.14.1 Sample Calculations

The next several pages present a sample calculation for a haunched, three-span, continuous flat-slab bridge.

16.3.14.2 Typical Details

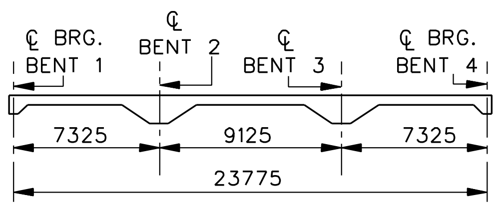
After the sample calculation, Figures 16.3C and 16.3F present details for MDT's typical half longitudinal slab sections and transverse slab sections.

Haunched, 3-Span, Continuous Flat-Slab Bridge

Given: Total Length = 23 775 mm
Roadway Width = 8500 mm
Rail Type: T101

Main-Span Length, L = 9125 mm
End-Span Length, 0.8L = 7325 mm (Section 16.3.1.3)

Sketch:

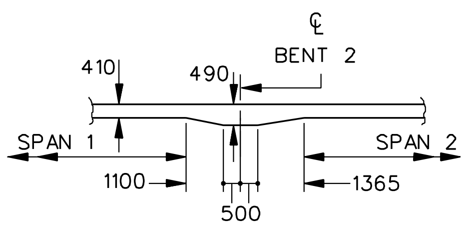


Minimum Slab Thickness = 410 mm (Section 16.3.10)

Haunch Thickness ≅ $1.2t_{min}$ = 490 mm (Section 16.3.1.3)

Length of Haunch ≅ 0.15L Main Span: 1365 mm (Section 16.3.1.3)
End Span: 1100 mm

Sketch:



Equivalent Strip Width (LRFD Article 4.6.2.3)

$$E = 250 + 0.42\sqrt{L_1 W_1} \qquad \text{(LRFD Equation 4.6.2.3} - 1\text{) for single-lane loaded}$$

$$E = 2100 + 0.12\sqrt{L_1 W_1} \le \frac{W}{N_L} \qquad \text{(LRFD Equation 4.6.2.3} - 2\text{) for multilanes loaded}$$

W = 8500 + 700 = 9200
N_L = 8500/3600 = 2.36 ≥ 2 (LRFD Article 3.6.1.1.1)
L_1 = 7325 mm (shorter span will control)

$$W_1 = 8500 + 700 = 9200 \text{ mm} \le \left| \begin{array}{l} \text{9000 mm for single-lane loaded} \\ \\ \text{18 000 mm for multilanes loaded.} \end{array} \right.$$

$$E_{single} = 250 + 0.42\sqrt{(7325)(9000)} = 3660 \text{ mm}$$

$$E_{multi} = 2100 + 0.12\sqrt{(7325)(9000)} = 3085 \text{ mm}$$

$$\frac{W}{N_L} = \frac{9200}{2} = 4600 \text{ mm}$$

Therefore, E = 3085 mm (Interior Strip)

Equivalent Strip at Edge of Slab (LRFD Articles 4.6.2.1.4b and 9.7.1.4)

$$E = 350 + 300 + \frac{1}{2}(3085)$$

$$= 2192.5 \text{ mm, but not exceeding } 3085 \text{ or } 1800.$$

$$\therefore \text{ use } E = 1800 \text{ mm } (\text{Edge Beam})$$

Dead Load

Interior Strip: $\left(\frac{3085 \text{ mm}}{1000}\right)\left(\frac{410 \text{ mm}}{1000}\right)(2400 \text{ kg/m}^3) = 3036 \text{ kg/m}$

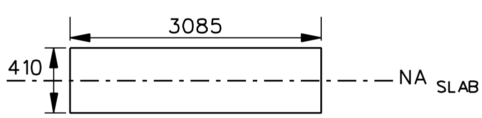
$= 29.8 \text{ kN/m}$

Edge Beam: $\left(\frac{1800 \text{ mm}}{1000}\right)\left(\frac{410 \text{ mm}}{1000}\right)(2400 \text{ kg/m}^3) = 984 \text{ kg/m}$

$= 9.7 \text{ kN/m}$

Modeling Section Properties for BTBEAM

Slab Section

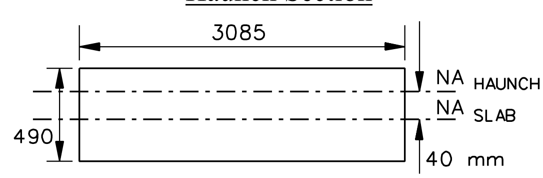


$$I = \frac{bh^3}{12}$$

$$= \frac{(3085)(410)^3}{12}$$

$$= 1.77 \times 10^{10} \text{ mm}^4$$

Haunch Section



$$I = \frac{bh^3}{12} + Ad^2$$

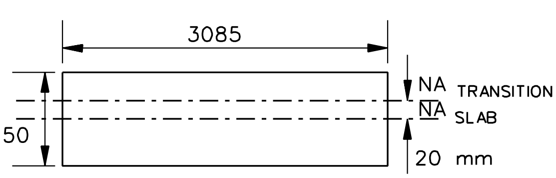
$$= \frac{(3085)(490)^3}{12} + 490(3085)(40)^2$$

$$= 3.02 \times 10^{10} + 2.42 \times 10^9$$

$$= 3.27 \times 10^{10}$$

Slab Transition Section

$$t_{avg} = \frac{410 + 490}{2} = 450$$

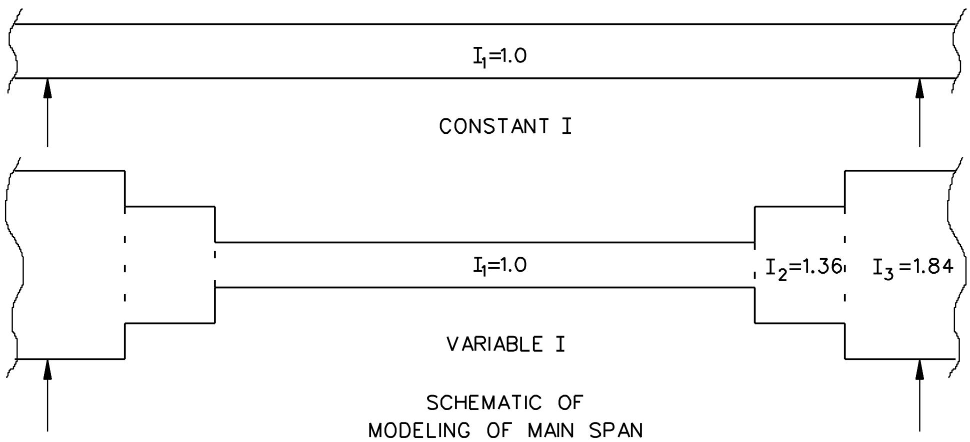


$$I = 2.34 \times 10^{10} + 5.56 \times 10^8$$

$$= 2.40 \times 10^{10}$$

BTBEAM Model

$I_1 = I_{slab} / I_{slab} = 1.0$
$I_2 = I_{tansition} / I_{slab} = 1.36$
$I_3 = I_{haunch} / I_{slab} = 1.84$



SCHEMATIC OF
MODELING OF MAIN SPAN

Results of BTBEAM Analysis of Interior Strip

Strength I Moments	Constant I	Variable I	Refined
End span	876 kN • m	847 kN • m	838 kN • m
	-893 kN • m	-988 kN • m	-1013 kN • m
Main span	880 kN • m	818 kN • m	800 kN • m

Discussion: The "Refined" analysis included the moment of inertia (I) at several sections along the haunch in the BTBEAM model. The results of the three methods of analysis indicate that the Constant "I" model grossly underestimates the negative moment at the supports compared to the Refined model. The results of the Variable "I" model and the Refined model are within 5% of each other, which is an acceptable tolerance for this design. Therefore, the Variable I method is recommended.

Design of Rectangular Flexural Sections

Design the typical cross sections at the point of maximum positive moment (mid-center span) and maximum negative moment (interior support) using LRFD Article 5.7.3 as appropriate.

$$\sum \gamma_i M_i = 847 \text{ kN} \bullet \text{m on } 3085 - \text{mm Interior Strip (maximum positive)}$$

or

$$\frac{847 \text{ kN} \bullet \text{m}}{3.085 \text{ m}} = 275 \text{ kN} \bullet \text{m} / \text{m}$$

$$\sum \gamma_i M_i = -988\ \text{kN} \bullet \text{m on } 3085 - \text{mm Interior Strip (maximum negative)}$$

or

$$\frac{-988\ \text{kN} \bullet \text{m}}{3.085\ \text{m}} = 321\ \text{kN} \bullet \text{m/m}$$

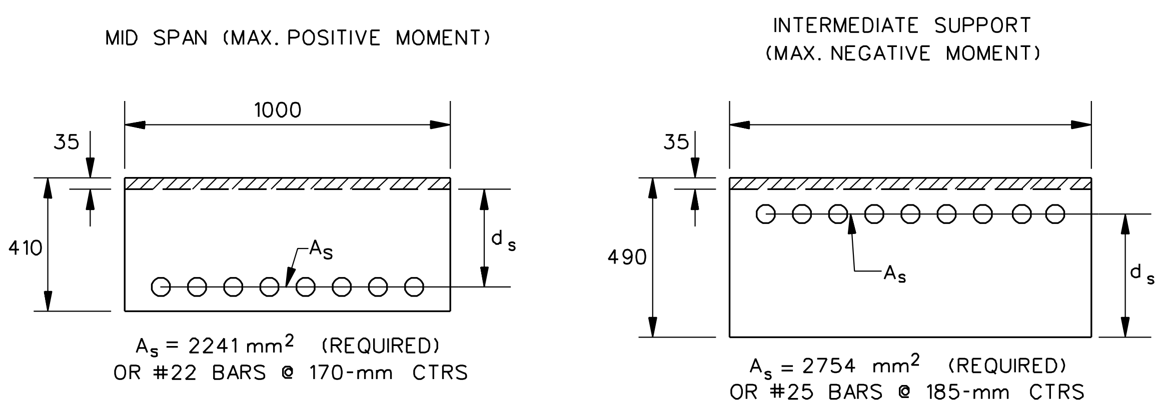
$$M_r = \varphi M_n \qquad \text{(LRFD Equation 5.7.3.2.1-1)}$$

$$M_n = A_s f_y \left(d_s - \frac{a}{2} \right) \qquad \text{(LRFD Equation 5.7.3.2.2-1)}$$

$$a = \beta_1 C \qquad \text{(LRFD Article 5.7.2.2)}$$

$$c = \frac{A_s f_y}{0.85\, f'_c \beta_1 b} \qquad \text{(LRFD Equation 5.7.3.1.2-4)}$$

Cross section



Transverse Distribution Reinforcement

Bottom of Slab (LRFD Article 5.14.4.1):

$$\frac{1750}{\sqrt{L}} \leq 50\% \qquad \text{(Maximum spacing (Article 5.10.3.2) = 450 mm)}$$

$$L = 7325\ \text{mm} \qquad \text{(Shorter span will control)}$$

$$\frac{1750}{\sqrt{7325}} = 20.5\% \qquad (A_s - 2241\ \text{mm}^2\ (0.205) = 459\ \text{mm}^2\text{, or \#16s @ 450-mm centers})$$

Shrinkage and Temperature Reinforcement

Top and Bottom of Slab (LRFD Article 5.10.8):

$A_s \geq 0.75\ A_g / f_y$

$$\geq \frac{0.75\,(1000)(410)}{420} = 732\ \frac{mm^2}{m}$$ (Equally distributed on top and bottom faces of slab in each direction.)

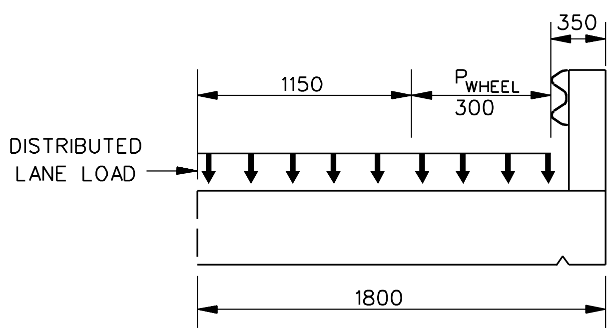
Maximum spacing = 3(410 mm) = 1230 mm or 450 mm

A_s = 366 mm^2 or #16s @ 450-mm centers (Top of slab, transverse)

Note: Shrinkage and temperature steel requirements, for top longitudinal, bottom longitudinal and bottom transverse, already satisfied by flexural and distribution requirements checked previously.

Analysis of Edge Beam

Sketch:



The BTBEAM live load analysis results are for an entire lane. Reduce the truck load results by ½ to get results for one wheel line, and reduce the redistributed lane load by 1800/3600 to determine the load on the notional edge beam.

Design of Rectangular Flexural Sections

BTBEAM analysis results for the edge beam using the Variable I method:

Strength I	End span: + 440 kN • m	
Moments	- 432 kN • m	(on 1800-mm strip)
	Main span: + 424 kN • m	

$$\sum \gamma_i M_i = \frac{440\ \text{kN} \bullet \text{m}}{1.800\ \text{m}} = 244\ \text{kN} \bullet \text{m} / \text{m} \quad \text{(maximum positive)}$$

$$\sum \gamma_i M_i = \frac{-432\ \text{kN} \bullet \text{m}}{1.800\ \text{m}} = 240\ \text{kN} \bullet \text{m} / \text{m} \quad \text{(maximum negative)}$$

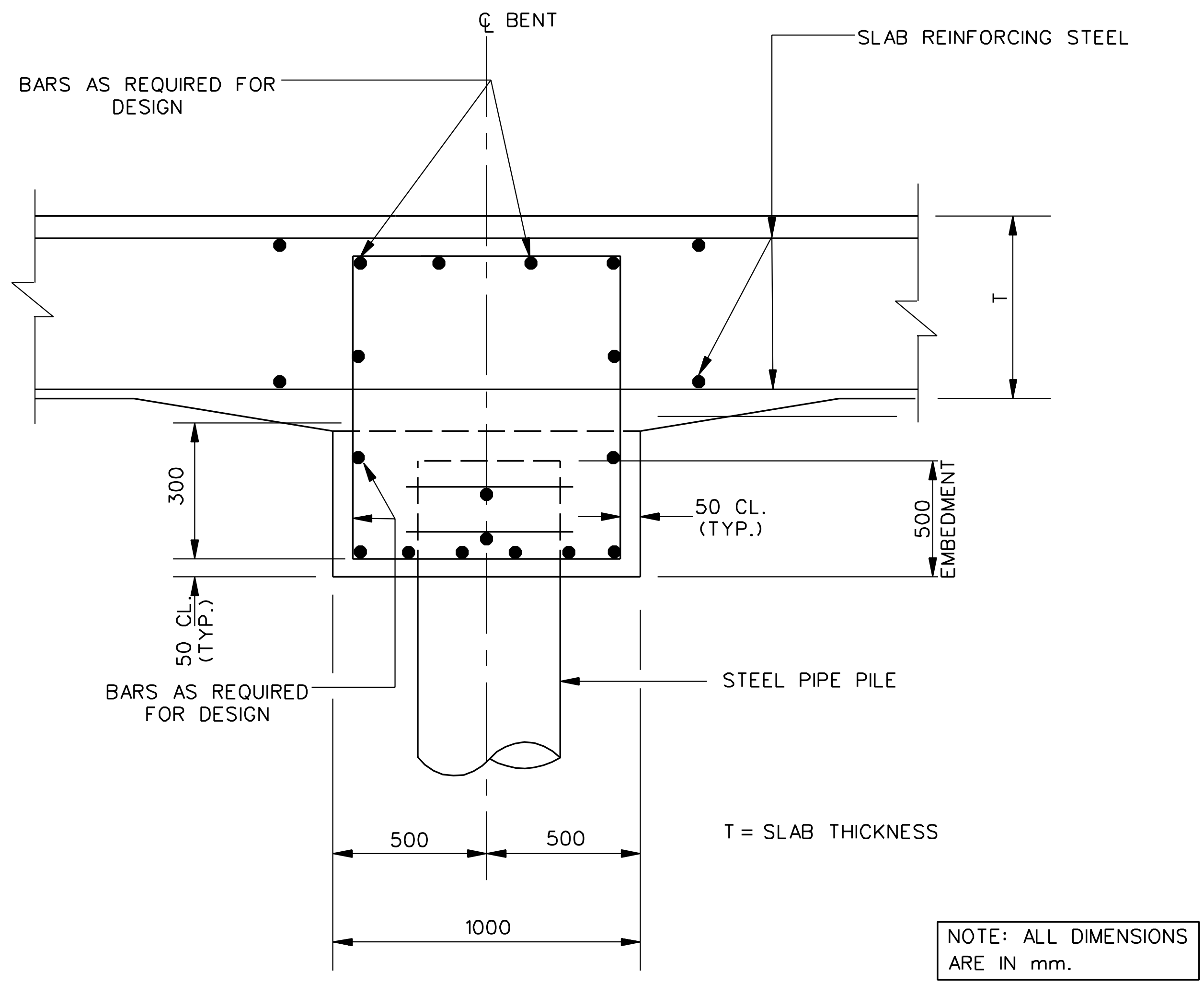
See Figure 16.3E for typical cross section.

$A_s = 3627\ mm^2$ (required for maximum positive moment)

or 8 #25s top and bottom.

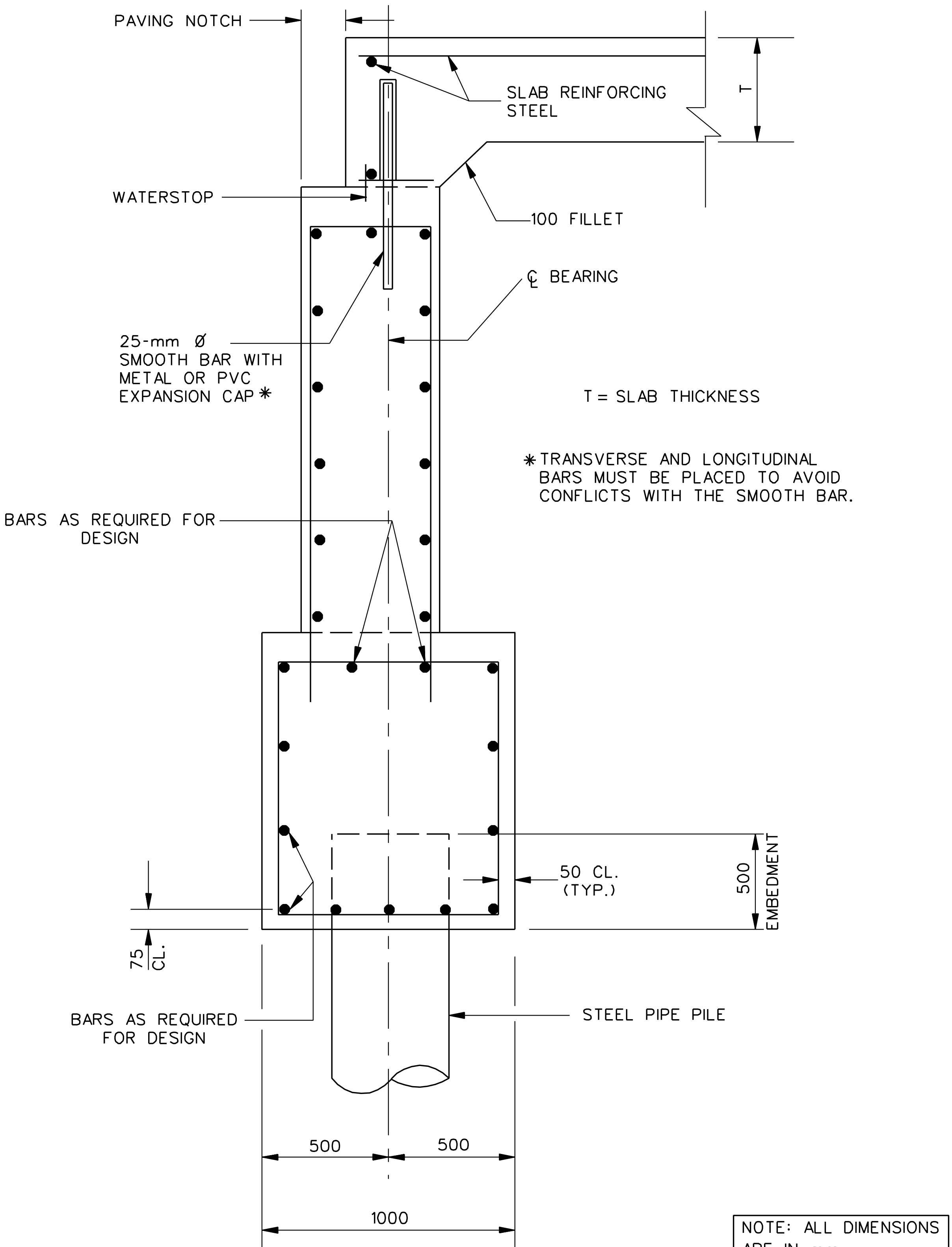
Shear Design of Edge Beam

For a constant-depth slab and integrated edge beam, shear reinforcing will not be necessary per Article 5.14.4.1. In special circumstances where a thickened edge beam is provided, shear should be investigated as set forth in Article 5.8.



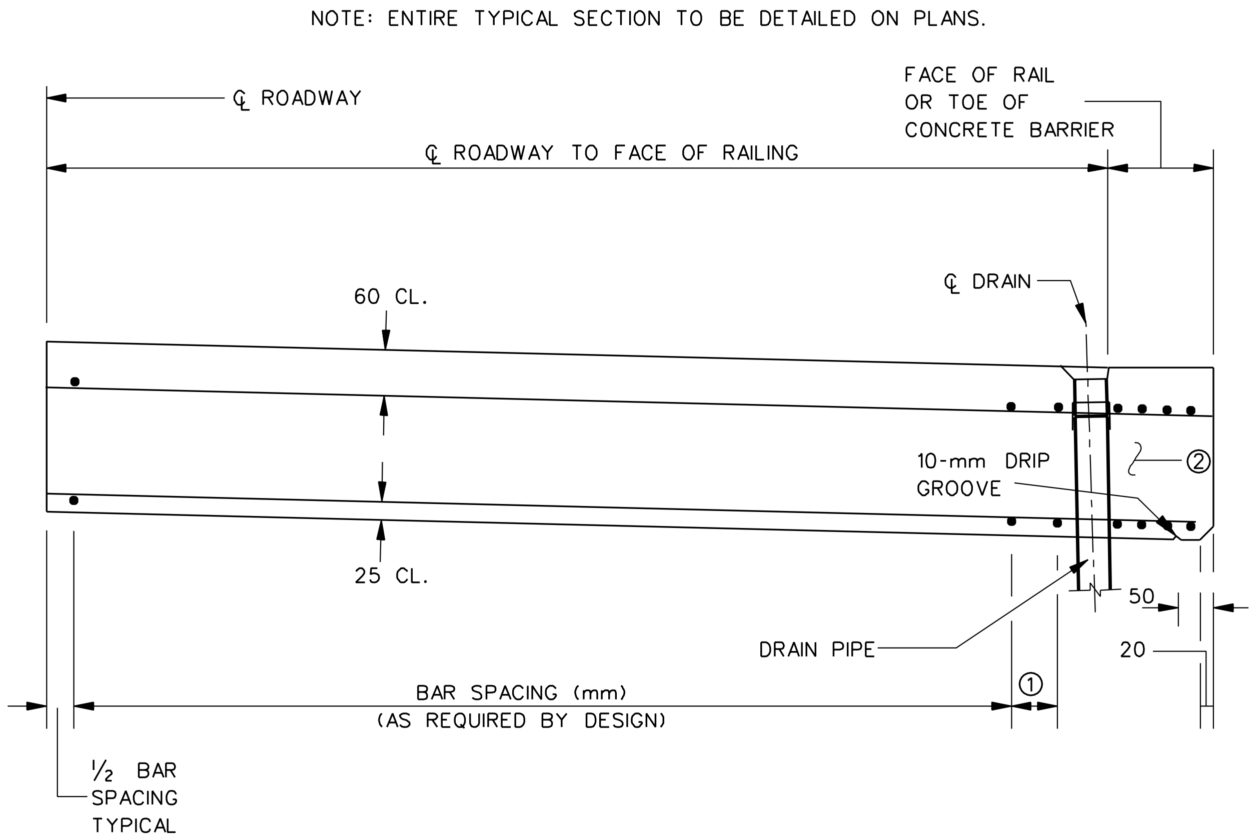
INTEGRAL CAPS AT INTERMEDIATE BENTS OF FLAT SLABS

Figure 16.3C



END BENTS OF FLAT SLABS
(Section Through End Bent)

Figure 16.3D



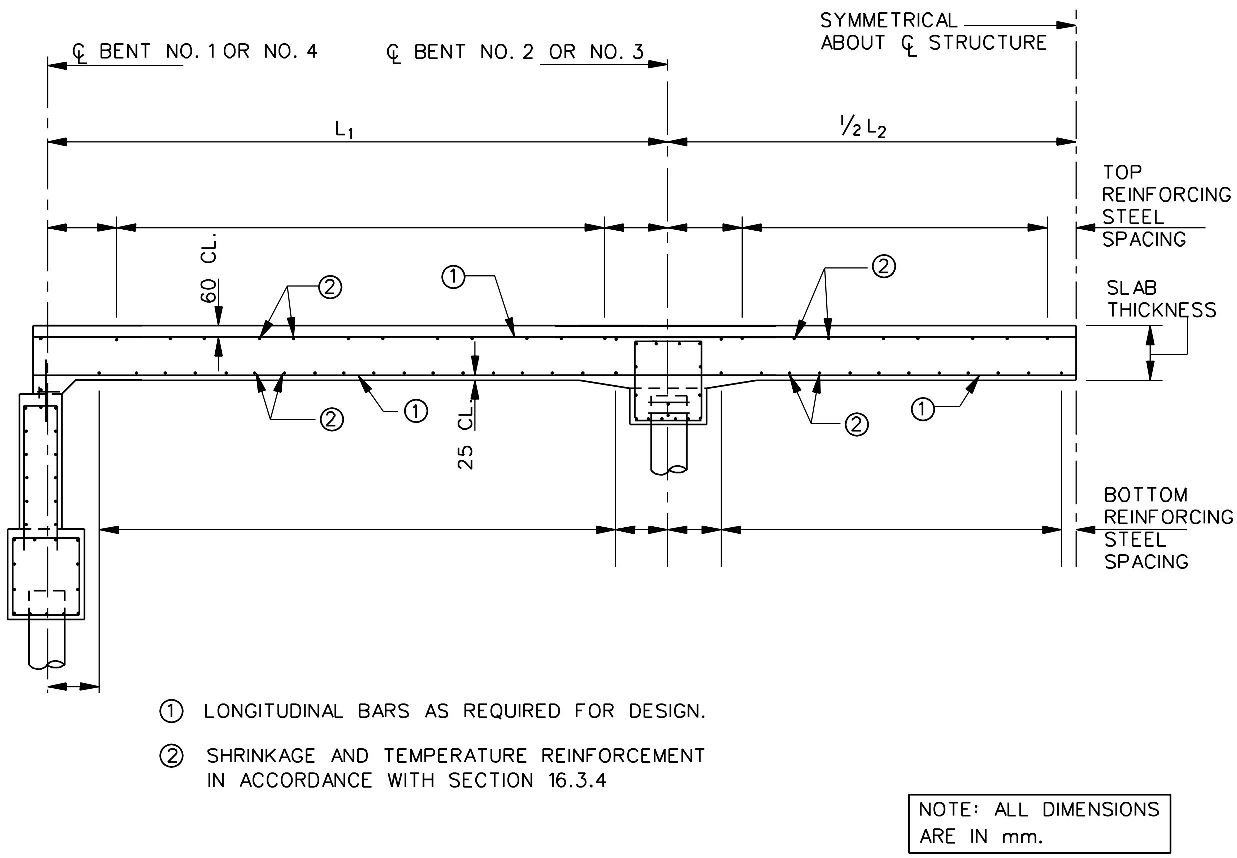
① BAR SPACING AND NUMBER OF SPACES TO BE DETERMINED TO FACILITATE A CONSTANT BAR SPACING IN REMAINDER OF SLAB.

② DESIGN EDGE BEAM IN ACCORDANCE WITH ARTICLES IN THE LRFD SPECIFICATIONS, BUT USE AS A MINIMUM THE SAME AREA OF STEEL PER METER AS IN SLAB.

NOTE: ALL DIMENSIONS ARE IN mm.

EDGE BEAM DETAIL, DECK SECTION OF FLAT SLABS
(Typical Half Section)

Figure 16.3E



INTEGRAL CAPS AT FLAT SLABS
(Half Longitudinal Section)

Figure 16.3F