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8 CONCRETE STRUCTURES

8.4 MATERIALS

8.4.1 Concrete

8.4.1.2 Concrete Strength

Insert after first sentence:

The specified concrete strength for prestressed members shall not exceed 55 MPa at 28 days or 37.5 MPa at release.

8.4.2 Reinforcing Bars and Deformed Wire

8.4.2.1 Reinforcing Bars

Reinforcing bar layouts shall be based on standard reinforcing bar lengths of 12 m for 10M bars and 18 m for 15M bars and greater.

Commentary: *Standard reinforcing bar lengths are based on typical bar lengths which are available from reinforcing steel suppliers.*

8.4.2.1.3 Yield Strength

Grade 400W reinforcing bars shall be specified for flexural reinforcement in plastic hinge regions.

Commentary: *Use of Grade 400W bars is intended to ensure plastic hinge regions possess expected ductility characteristics.*

For Grade 400W reinforcing bars, an upper limit for yield strength of 525 MPa is a requirement of CAN/CSA-G30.18.

8.7 PRESTRESSING REQUIREMENTS

8.7.4 Loss of Prestress

8.7.4.1 General

Commentary: *The designer is cautioned that the losses tabulated in Table C8.7.4.1 may be unconservative for prestressed girders where the span to depth ratio pushes the capacity limit of the section.*

8.8 FLEXURE AND AXIAL LOADS

8.8.4 Flexural Components

8.8.4.5 Maximum Reinforcement

The requirement of this clause may be waived by the design engineer provided it is established to the satisfaction of the Ministry that the consequences of reinforcement not yielding are acceptable.

8.9 SHEAR AND TORSION

8.9.3 Sectional Design Model

Commentary: *Design for seismic shear based on S6-00 within ductile sub-structures does not address shear resistance within a plastic hinge zone. Recent design standards or model standards, such as ATC-32 and ATC-49, as well as the current Caltrans Seismic Design Criteria, attribute higher shear capacities than S6-00 in non-ductile regions of reinforced concrete bridge columns, but lower shear capacities in plastic hinge regions. While designers are encouraged to adopt state-of-the-art seismic design methods for shear, care is required to achieve an appropriate margin of safety against brittle failure modes. The responsibility for achieving an acceptable margin of safety is the responsibility of the designers.*

The following approach comprises an acceptable design method for shear resistance within ductile concrete sub-structures, and provides a minimum design shear resistance within plastic hinge zones.

Seismic shear resistance within reinforced concrete columns of ductile sub-structures may be taken as follows:

$$\Phi V_n = \Phi_c V_c + \Phi_s V_s + \Phi_s V_p$$

Note that Φ_s is suggested for the V_p term where the column axial load is dominated by gravity loads and flexural hinging in the ductile substructure. The designer should consider using a lower Φ value where conditions warrant.

The various terms of the General Method for shear design are similar to those commonly found in state of the art references and guide standards on seismic design of bridges. Examples of different approaches for each term are provided below. Selected references for shear design of bridge columns include:

- *Priestley, M.J.N., Verma, R., and Xiao, Y., 1994, Seismic Shear Strength of Reinforced Concrete Columns, Journal of Structural Engineering, 120 (8), pp. 2310-2329.*

- *Kowalsky, M.J. and Priestley, M.J.N., 2000, "Improved Analytical Model for Shear Strength of Circular Reinforced Concrete Columns in Seismic Regions." ACI Structural Journal, Vol. 97, No. 3 pp. 388-396, May.*
- *Sezen, H., and Moehle, J.P., 2004, "Shear Strength Model for Lightly Reinforced Concrete Columns," Journal of Structural Engineering, Vol. 130, No. 11, pp. 1692-1703.*
- *Camarillo, H.R., 2003, "Evaluation of Shear Strength Methodologies for Reinforced Concrete Columns," M.S. Thesis, Department of Civil and Environmental Engineering, University of Washington.*
- *ATC-32 Section 8.16.6 (ATC-32 were provisional recommendations to Caltrans, and may be regarded as superseded by subsequent Caltrans Seismic Design Criteria).*
- *Seismic Design and Retrofit of Bridges, Priestley and Calvi (1996).*
- *Caltrans Seismic Design Criteria (latest version, currently Ver. 1.3, 2004), Section 3.6.*
- *ATC-49 Section 8.8.2.3.1 and 8.8.2.3.2.*

8.9.3.4 Determination of V_c

8.9.3.4.1 General Method

Commentary: *β may be taken as 0.29 for columns of ductile sub-structures of nominal ductility structures, and not less than 0.05 for plastic hinge regions of columns of ductile sub-structures, or where curvature ductilities are not determined. Interpolation between these two values for curvature ductilities between 3 and 15 may be used.*

This approach to calculate β, the concrete shear contribution in plastic hinge zones, is based on Priestley et all (2000) and is similar to the approach in ATC-49.

8.9.3.5 Nominal Shear Stress

Commentary: *For the seismic design of round reinforced concrete columns or piers, the effective shear area shall be equal to 80% of the gross concrete area, A_g.*

8.9.3.6 Determination of ε_x

Commentary*:* *For the design and evaluation of prestressed girders the capacity-enhancing effect of negative strains (compressive) near supports may be taken into account. Acceptable approaches can be found in the latest CSA A23.3 Standard or AASHTO LRFD specification.*

8.9.3.8 Determination of V_s

***Commentary**: S6-00 currently does not differentiate between rectangular and round columns for the determination of the tie or spiral reinforcing contribution to shear resistance. The formulae provided above is adopted from Priestley et al (2000), and comprises an acceptable method to calculate Vs as part of the seismic shear resistance of round columns. It may be used within our outside of plastic hinge zones. Designer must satisfy themselves that the column configuration, details and axial load under gravity in combination with seismic loads justify a value of θ = 30°. Higher angles (cracks crossing fewer spirals, hence lower V_c) should be used where warranted.*

$$V_s = \phi_s (0.5 \pi A_v f_{yh} D' \cot \theta) / s$$

Where:

A_v = area of one spiral bar

f_{yh} = yield stress of spiral reinforcing

D' = core diameter of the column, approximately equal to the diameter measured across the centre of the vertical reinforcing steel bar cage

θ = 30° for the purposes of this clause only, or 45° if axial tensile loads occur under seismic loads

s = spacing of spiral reinforcing bars

8.11 DURABILITY

8.11.2 Protective Measures

8.11.2.1 Concrete Quality

For structural elements listed below, concrete mix criteria shall be specified in the Special Provisions and shall comply with the requirements given in the following table unless otherwise consented to by the Ministry:

Classification	Minimum Compressive Strength at 28 days (MPa)	Nominal Maximum Size of Coarse Aggregate (mm)	Air Content (%)	Slump (mm)	Maximum W/Cm Ratio by Mass
Deck Concrete: Deck Slab, Approach Slab, Parapet and Median Barrier					
• Standard(4)	35	28(1)	5 ± 1	50 ± 20	0.38
• With Silica Fume	35	28(1)	6 ± 1	80 ± 20(2)	0.38
• With Class F or C1 Flyash(3,4)	35	28(1)	6 ± 1	50 ± 20	0.38
Substructure Concrete: Piers, Abutments, Retaining Walls, Footings, Pipe Pile Infills, Working Floors	30	28	5 ± 1	50 ± 20	0.45
Keyways between Box Stringers	35	14	5 ± 1	20 ± 10	0.38
Concrete Slope Pavement	30	20	5 ± 1	30 ± 20	0.45
Deck Overlay Concrete:					
High Density(4)	35	20(5)	5 ± 1	30 ± 20	0.38
Silica Fume Modified	45	14(6)	6 ± 1	60 ± 20(2)	0.38

Notes:

(1) The maximum proportion of aggregate passing the 5 mm screen shall be 35% of the total mass of aggregate.

(2) Silica fume application rates shall be 8% maximum by mass of Portland Cement. Slump specification is based on superplasticized concrete.

(3) Application rates shall not exceed 15% by mass of Portland Cement.

(4) Superplasticizer shall not be used.

(5) The maximum proportion of aggregate passing the 5 mm screen shall be 38% of the total mass of aggregate.

(6) The maximum proportion of aggregate passing the 5 mm screen shall be 42% of the total mass of aggregate.

The gradation of the 28 mm nominal size aggregate shall conform to Table 211-B in SS 211 unless noted otherwise in this clause.

8.11.2.1.2 Concrete Placement

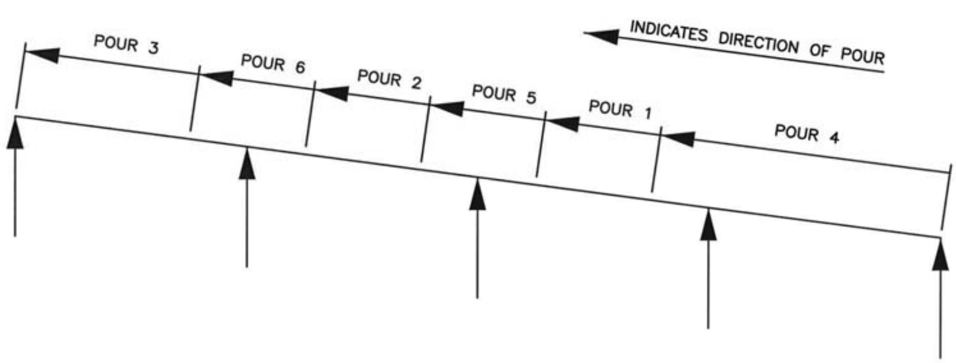
The deck casting sequence and the detail for construction joints shall be shown on the drawings. Typically, deck slabs shall be cast in the direction of increasing grade (uphill). Bridges with minimum grades may be cast in either direction. . Consideration shall be made in specifying the casting sequence to minimize the number of construction joints (cold joints) in the deck.

For simply supported span structures, each span shall be cast in one continuous operation unless otherwise consented to by the Ministry.

For continuous structures, concrete shall typically be cast as follows:

- Concrete in positive-moment zones: All concrete in these zones shall typically be cast prior to concrete in negative-moment zones.
- Concrete in negative-moment zones: Concrete in these zones shall typically not be cast until adjacent concrete in positive- moment zones has cured, unless cast monolithically with the positive-moment concrete as shown below.

Figure 8.11.2.1.2
Sample Schematic of Deck Pour Sequence
(to be shown on the drawings)



Concrete placing sequence for integral abutments shall be given special consideration to reduce stresses induced by deflection of the girders.

Commentary: *A deck casting sequence is required in order to minimize the potential for deck cracking due to improper concrete placement sequencing.*

Several factors limit the quantity of concrete which can be placed in one continuous operation. Special consideration shall be given if the continuous placement exceeds a volume of 200 cubic metres or if the bridge deck exceeds four lanes in width.

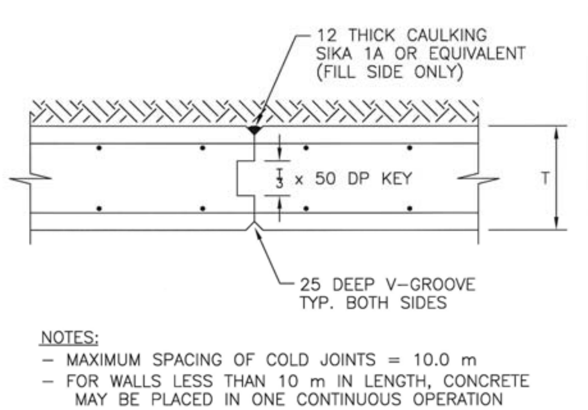
For continuous span bridges, the length of deck casting should be limited to 20 m in order to minimize shrinkage cracking.

For integral abutments, techniques for reducing stresses induced by deflection of the girders may include delaying the casting of the abutments and/or the deck in the abutment area until after all other deck concrete has been cast.

8.11.2.1.4 Cold Joints

A suggested detail for a typical cold joint in a wall is shown in Figure 8.11.2.1.4 below.

Figure 8.11.2.1.4 - Typical Cold Joint for Walls



8.11.2.1.5 Slip-Form Construction

Extruded concrete barriers shall not be used.

Commentary: *It has been observed that the use of extruded concrete barriers does not result in a watertight joint at the interface with the deck which allows seepage of water to cause staining of the deck and superstructure.*

8.11.2.1.6 Finishing

Surface finishes shall be in accordance with Table 8.11.2.1.6 and shall be specified in the Special Provisions.

Table 8.11.2.1.6

Surface	Finish	Standard Specification Clause
Surfaces submerged or buried	Class 1	211.17
Top and inside (exposed) face of parapets, curbs	Class 3	211.17
Outer face of parapets, curbs; outer edges of deck	Class 2	211.17
Abutments and retaining walls	Class 2	211.17
Piers	Class 2	211.17
Bearing seats	Steel Trowel	211.14
Top of deck	Tined(2)	413.31.02.05
Approach slabs	Float Finish	211.14
Sidewalks	Transverse Coarse Broom	211.14
Underside of Deck	Class 1 (or better)	211.17
Slope Pavement	Transverse Coarse Broom(1)	2111.14

Notes

(1) Exposed Aggregate finishes may be considered.

(2) Decks to receive waterproofing membranes shall be finished in accordance with Standard Specification 419.33.

Consideration shall be given to surfaces exposed to public view such as piers and abutments on underpasses where a Class 3 finish may be warranted, and underside of decks where a Class 2 finish may be warranted.

Exposed concrete surfaces of large abutments or retaining walls that are clearly visible to the public may require an architectural finish. The selection of a surface finish shall give consideration for future removal of graffiti. Such consideration may include the application of anti-graffiti paint

8.11.2.2 Concrete Cover and Tolerances

The soffits of deck slabs cantilevered from the exterior girder shall be considered under Environmental Exposure Class (a) while soffits of deck slabs intermediate to the exterior girders may be considered under Environmental Exposure Class (b) as detailed in Table 8.11.2.2.

All references to "minimum cover" in S6-00 shall be replaced with "minimum specified cover". Table 8.11.2.2 in S6-00 shall be amended as follows:

8.11.2.2	**Minimum Specified Concrete Covers and Tolerances**			
			Concrete Covers and Tolerances	
Environmental Exposure	**Component**	**Reinforcement/ Steel Ducts**	**Cast-in-Place Concrete (mm)**	**Precast Concrete (mm)**
All	(3) Top surfaces of Structural components			
	Add: Bridge Decks and Approach Slabs	**Reinforcing Steel**	**70 +6 -0**	as per Table 8.11.2.2 of S6-00
All	(10) Precast T, I and box girders			
	Add: Ministry Standard Precast Box Girders	**Reinforcing steel**		
		- Top surfaces	–	**70 +10 -5**
		- Vertical surfaces	–	**40 +10 -5**
		- Soffits	–	**30 +10 -5**
		- Inside surfaces	–	**30 +10 -5**
		Pretensioning strands		
		- Top surfaces	–	**200 ±5**
		- Vertical surfaces	–	**50 ±5**
		- Soffits	–	**40 ±5**
		- Inside surfaces	–	**35 ±5**
	Add: Ministry Standard Precast I-Beams	**Reinforcing steel**		
		- Top surfaces	–	**30 +10 -5**
		- Vertical surfaces	–	**30 +10 -5**
		- Soffits	–	**30 +10 -5**
		Pretensioning strands		
		- Top surfaces	–	**100 ±5**
		- Vertical surfaces	–	**40 ±5**
		- Soffits	–	**40 ±5**

Delete Note *** under Table 8.11.2.2. An additional 10 mm of concrete cover shall not be provided for concrete decks without waterproofing and paving.

Commentary: *The term "minimum cover" should be avoided as it creates confusion for installers. The term "specified cover" is the preferred term and the appropriate placing tolerances would apply. For top reinforcing in decks, a "specified cover" of 70 mm with placing tolerances of +6 mm and -0 mm will provide the correct installation.*

Designers must be aware of, and account for, placing tolerances and specified cover requirements. As an example, consideration shall be given to the cover requirements on mechanical splices.

8.11.2.3 Corrosion Protection for Reinforcement, Ducts And Metallic Components

As a minimum, the top mat of deck reinforcing steel, all reinforcing steel in cast-in-place parapets and reinforcing steel in approach slabs shall be protected against corrosion.

Epoxy coating or galvanizing may be used for corrosion protection of reinforcing steel. If galvanizing is used, all reinforcing steel in the component shall be galvanized. Galvanized bars and uncoated bars shall not be used in combination in any one structural component.

Stainless steel may be considered as an alternative to epoxy coating or galvanizing if strength requirements are met and its use is found to be comparatively economical. Stainless steel clad reinforcing may only be used with consent of the Ministry.

Ends of prestressing strands shall be painted with a Ministry accepted organic zinc rich paint where the ends of stringers are incorporated into concrete diaphragms or are otherwise embedded in concrete.

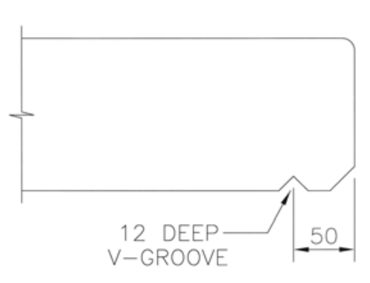
Ends of prestressing strands shall be given a minimum 3 mm coat of thixotropic epoxy in 100 mm wide strips applied in accordance with the manufacturer's requirements where ends of stringers are not embedded in concrete.

Commentary: *Galvanized reinforcing steel and uncoated steel should not be used in combination due to the possibility of establishing a bimetallic couple between zinc and bare steel (i.e. at a break in the zinc coating or direct contact between galvanized steel and black steel bars or other dissimilar metals).*

8.11.2.6 Drip Grooves

Continuous drip grooves shall be formed on the underside of bridge decks and shall be detailed as shown below in Figure 8.11.2.6

Figure 8.11.2.6
Drip Groove Detail



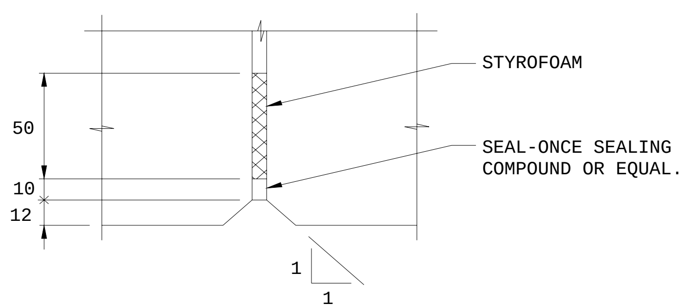
Commentary: *The drip groove detail shown above has been used throughout the Province since it was first introduced in September 1989 and has functioned well with no adverse feedback from field staff. For this reason the detail has been retained, although it varies from the drip groove detail shown in Clause 8.11.2.6 of S6-00.*

8.12 CONTROL OF CRACKING

8.12.1 General

Control joints shall be evenly spaced through the length of the barrier and their spacing shall not exceed 5 m. They shall extend around the perimeter of the barrier

Concrete traffic barriers shall have a 6 mm joint over the supports on continuous spans. The joints may be saw-cut, but the structure shall not be subjected to a single vehicle live load greater than 5 Kn prior to the cutting operation.



8.13 DEFORMATION

8.13.3 Deflections and Rotations

8.13.3.3 Total Deflection and Rotation

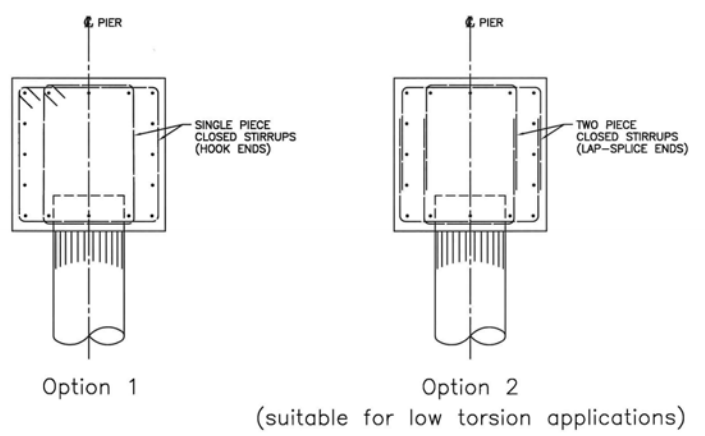
Commentary: *The Commentary to S6.00 states that long time deflection and rotation may be calculated by using the empirical multipliers given in Table C8.13.3.3 which is taken from CPCI (1996). However, Table C8.13.3.3 is not an exact copy of the table included in CPCI (1996). The original table may be used in place of the commentary.*

8.14 DETAILS OF REINFORCEMENT AND SPECIAL DETAILING PROVISIONS

8.14.3 Transverse Reinforcement for Flexural Components

Typical arrangements for transverse reinforcement of pier caps are shown in Figure 8.14.3.

Figure 8.14.3
Typical Transverse Reinforcement of Extended Pile Pier Caps



Commentary: *The typical transverse reinforcement arrangements shown in Figure 8.14.3 alleviate problems encountered with installation of longitudinal reinforcing in situations where piles are installed slightly off alignment. These preferred arrangements facilitate placement of two longitudinal bars in close proximity to the piles. Identical-size pairs of closed stirrups which lap one another horizontally do not provide as much tolerance for placement of the two longitudinal bars adjacent to the piles.*

For diaphragms and other varying depth members, closed stirrups formed from two piece lap-spliced U-stirrups are preferred.

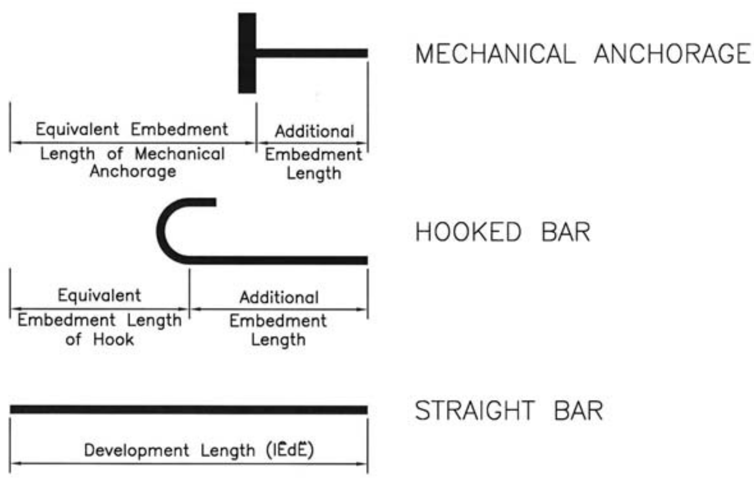
Commentary: *Problems are encountered with stirrup sizes in diaphragms when stirrups are either too long or too short depending on the final depth of the haunches. The method of using two piece U-stirrups of suitable depth alleviates problems in accommodating variations in depth of diaphragms.*

8.15 DEVELOPMENT AND SPLICES

8.15.6 Combination Development Length

Figure 8.15.6 below illustrates how the development length, l_d, may consist of a combination of the equivalent embedment length of a hook or mechanical anchorage plus additional embedment length of the reinforcement measured from the point of tangency of the hook.

Figure 8.15.6
Combination Development Length



8.15.9 Splicing of Reinforcement

All splices that are critical to the structure shall be indicated on the Drawings.

Splicing of transverse reinforcing bars in bridge decks shall be avoided. If such splices are unavoidable, their location shall be indicated on the Drawings.

8.16 ANCHORAGE ZONE REINFORCEMENT

8.16.7 Anchorage of Attachments

Dowel holes for Ministry standard prestressed concrete box stringers shall be detailed as shown on the Ministry standard reference details (Ministry Standard Drawings 2978-1B to 2978-24B) for box stringers and further described as follows:

a) Fixed Joint

Dowel holes shall be filled with non-metallic, non-shrink grout with strength as specified by the designer.

b) Expansion Joint

Dowel holes and dowels shall be sized to ensure there is adequate space for horizontal movement and also vertical movement if required for jacking.

8.18 SPECIAL PROVISIONS FOR DECK SLABS

Bridge deck heating systems shall not be incorporated into the design of bridge decks.

Commentary: *Heating of bridge decks in British Columbia has been problematic. Its use has therefore been discontinued.*

8.18.2 Minimum Slab Thickness

A minimum deck slab thickness of 225 mm shall replace the minimum slab thickness of 175 mm required in Clause 8.18.2 of S6-00.

Commentary: *The minimum deck slab thickness is based on providing adequate clear concrete cover between top and bottom layers of deck reinforcement and maintaining top and bottom concrete covers for the deck slab.*

Concrete cover – top of deck	*70 + 6 mm (tolerance)*
Top reinforcing – transverse	*18 mm*
Top reinforcing – longitudinal	*18 mm*
Minimum clear cover between layers	*25 mm*
Bottom reinforcing – longitudinal	*18 mm*
Bottom reinforcing – transverse	*18 mm*
Concrete cover – soffit of deck	*40 + 10 mm (tolerance)*
Total - Minimum slab thickness	*223 mm (round up to 225 mm)*

8.18.3 Allowance for Wear

Delete this clause.

8.18.5 Diaphragms

Add the following sentence to the end of the first paragraph:

Intermediate steel diaphragms shall be used on prestressed concrete girders which are not skewed.

For monolithic cast-in-place concrete end diaphragms and intermediate diaphragms, consideration shall be given to additional deck reinforcing over the diaphragms to withstand negative moment demands. Refer to Clause 8.20.8 for specific guidance regarding design of concrete diaphragms for concrete girders.

8.19 COMPOSITE CONSTRUCTION

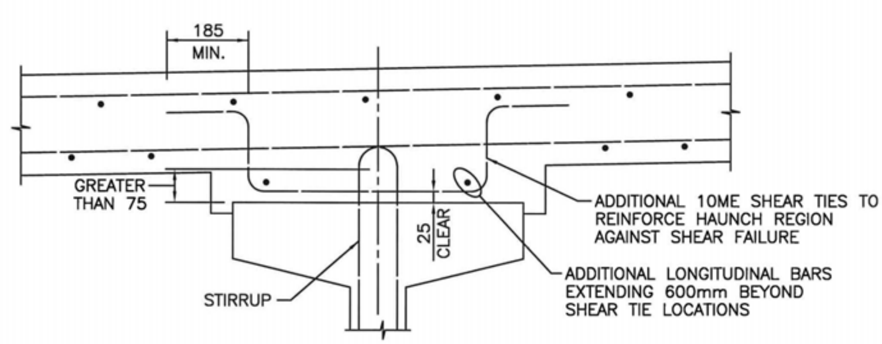
8.19.1 General

Prestressed concrete box girders with a concrete overlay shall be designed as non-composite unless mechanical anchorage is incorporated to ensure composite action. For non-composite design, the placement of a concrete overlay on top of box girders shall be considered as an additional dead load and shall not be assumed to contribute to any composite properties under live loads.

8.19.3 Shear

Shear reinforcement in prestressed I-beams shall extend 125 mm above the top of the beam. When the haunch height exceeds 75 mm, additional shear reinforcement (e.g. shear ties matching the spacing of stirrups in the I-beams) and additional longitudinal reinforcing at the haunch corners shall be provided as shown in Figure 8.19.3.

Figure 8.19.3
Additional Reinforcement for Haunches over 75 mm High



Additional shear reinforcement and longitudinal reinforcing at the haunch corners shall also be provided above steel girders where haunch heights exceed 75 mm. Refer to Clause 8.11.2.3 regarding use of galvanized reinforcing bars.

8.20 CONCRETE GIRDERS

8.20.1 General

Prestressed concrete I-girder and box girder skews over 30° shall be avoided where practical. Where skews over 30° are used, sharp corners at ends of girders shall be chamfered as a precaution against breakage.

Box girders shall be skewed in increments of 5°.

8.20.3 Flange Thickness for T- and Box Girders

8.20.3.2 Bottom Flange

Ministry Standard Twin Cell Box Stringers shown on Drawings 2978-1B to 2978-24B shall be used as Ministry standards for twin cell boxes.

Commentary: *The bottom flange thickness of Ministry standard prestressed concrete box stringers does not comply with the minimum code requirement of 100 mm. No rationale is given in the Code or the Commentary for this minimum requirement.*

The current series of standard twin cell boxes have been in use since the late 1970's and have performed extremely well over the years. The increase in cost of fabrication and transportation necessary to update to the cover requirements of S6-00 is not considered to be warranted.

8.20.4 Web Thickness

Table 8.20.4: Minimum Web Thickness

Amend the web thickness value for pretensioned precast I-girders to 127 mm from 160 mm.

Commentary: *Although the Ministry standard prestressed I-beam sections with 127 mm thick web do not meet the code minimum requirement of 160 mm, they have functioned well since they were introduced in 1987. The 127 mm thick web has not resulted in any problems to fabricators in placing of prestressed and non-prestressed reinforcement. An increase in web thickness to meet code requirements will result in transportation limitations for Type 5 and higher girder types and increased cost of production.*

For post-tensioned construction, the web thickness shall be increased to satisfy code requirements for cover and tolerances.

8.20.7 Post-Tensioning Tendons

Unbonded post-tensioning tendons shall not be used.

Commentary: *Unbonded tendons have experienced numerous corrosion incidents due to inadequacies in corrosion protection systems, improper installation, or environmental exposure before, during and after construction.*

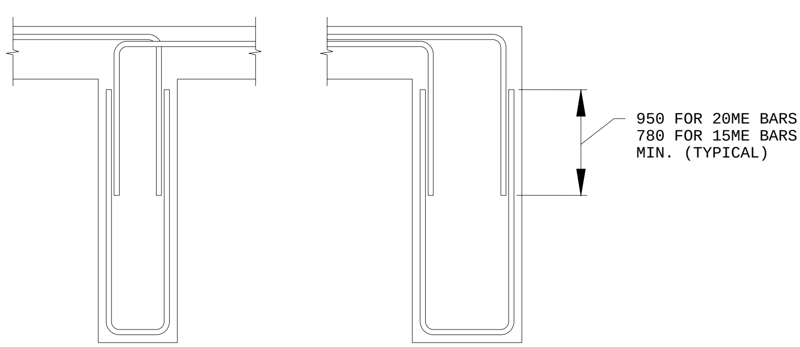
8.20.8 Diaphragms

Delete and replace with the following:

Diaphragms shall be provided at abutments and piers to support the deck and transfer loads to the supports. Abutment and pier end diaphragms shall be oriented parallel to the bridge skew and shall have a minimum thickness of 350 mm. Additional reinforcing shall be placed between longitudinal temperature reinforcement to account for negative moment effects. The minimum added reinforcing shall be **15M** bars and shall extend for a distance S/2 into the deck slab from the edge of the diaphragm where 'S' is the c/c of stringers. The bars shall have a standard hook at the diaphragm end. Where intermediate diaphragms support the slab, bars shall be added between the longitudinal reinforcing. The bars shall be **15ME** and the length shall equal 'S.'

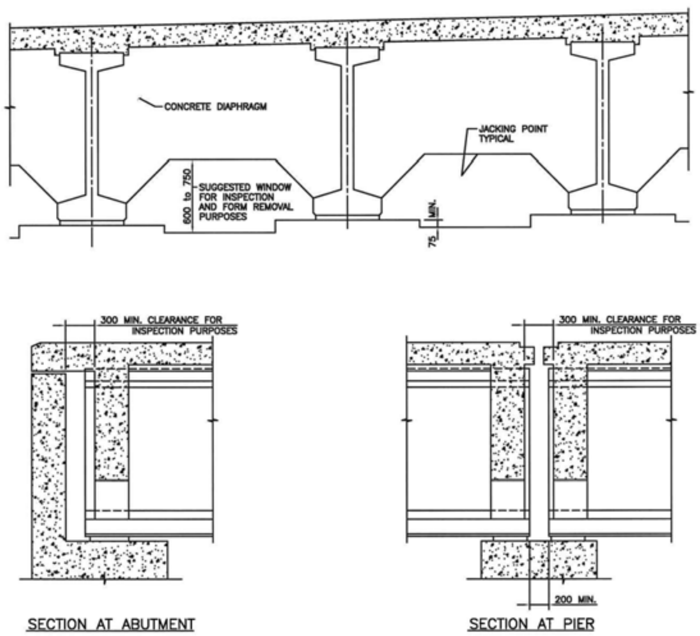
A typical tie arrangement for intermediate and end diaphragms is shown in Figure 8.20.8.1.

Figure 8.20.8.1 - Typical Diaphragm Tie Arrangement



Abutment and pier diaphragms shall be designed to facilitate future jacking, and to provide access for maintenance inspection, as generally outlined in Figure 8.20.8.2

Figure 8.20.8.2 – Typical Concrete Diaphragm Hole Sizes through Ends of Prestressed Girders



The hole size for abutment and pier diaphragm reinforcing which passes through the ends of prestressed girders shall be 2.5 times the bar diameter.

8.21 MULTIBEAM DECKS

The shear key and reinforcement details shown on Ministry Standard Drawings 2978-1B to 2978-24B (Twin Cell Concrete Box Girder) shall be considered as an Approved means for live load shear transfer between multibeam units in accordance with Clause 8.21(c) of S6-00.

Commentary: *Ministry standard box stringers less than 20 m in length without lateral post-tensioning have performed well (no longitudinal cracks or leaks) since they were first introduced in the late 1970's. According to recently completed site investigations by the Ministry on multi-beam decks with asphalt overlay where transverse post-tensioning was not used, no longitudinal cracking of the asphalt overlay was observed over shear key areas. The majority of the non-composite box spans investigated were less than 20 m spans.*

Standard box stringer bridges up to 30 m may also be used without lateral post-tensioning, provided explicit analysis indicates that the shear key has sufficient live load shear transfer capacity.

In most cases, a reinforced concrete overlay is applied as a wearing course topping on twin or single cell box beams. Where specified as an alternative to a concrete overlay, or as otherwise consented to by the Ministry, the top surfaces may be protected with a waterproofing membrane on the Ministry Recognized Products List, and applied in accordance with the manufacturer's instructions with an asphalt overlay of 100 mm placed in two lifts of 50 mm.