Ontario

Comments received by TCP (000-0184)			
Comment ID	**Organization**	**Comment**	**Response**
390	**Individual**	Structural manual should include provisions for designer to review and determine if large concrete components would potentially require thermal intervention during curing. This could include estimating mass concrete and thermal heat of hydration to determine if it is likely to exceed 70C and if so, add provisions to the contract. This could include planned and designed control/construction joints to limit the size of each component to allow for natural cooling between staged pours. include NSSP's allowing for different mix designs for footing elements (i.e.. Additional slag in components not exposed to de-icing salts) or cooling systems. Finally notes to the GA to account for any optional joints or cooling system(s). Notes on GA could include "Contractor is responsible for temperature control plan and thermal regulation system. - If installed, any temporary cooling tubing or system is to be placed inside the component as to no impact the reinforcing placement and locations. All temporary tubing is to be grouted once curing period is complete and no tube ends shall be left within 50mm of concrete surface. etc.	Thank you for your comment. We will add a statement to make the designer aware of situations where additional cooling measures may be needed. We will include suggested provisions in OPSS 904 & CDED 904, which will be updated in 2024.
392	**Senior Bridge Engineer/Project Manager EXP Service Inc.**	Thanks for your efforts and contribution for the new version of Structural Manual, look forward to using the new version in coming structural design.	Thank you for your comment.
393	**Individual**	2.6.1 Drawing Numbers It says what to do if the drawings are preliminary (P1), and it says what to do if the drawings are rehabs (R2-1). But, when I look at the structural sheets of new bridges they are commonly prefixed with "S" (e.g., S1). Should the manual not say that more clearly? Or is the intent to actually drop the "S" for the structural sheets?	The section specifies how drawing numbers, which appear in the bottom right hand side of the border, should be enumerated. According to MTO standards, they should not appear with a prefix 'S'. Sheet numbers appearing at the top right hand corner are specific to the contract and not covered by this section. An explanation has been added to this section.

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Comments received by TCP (000-0184)			
394	**Individual**	I think if Section 2.5.1 is moved to Section 5.4 so all requirements for RSS could be found in one place. Also, are there special requirements for submerged or partially submerged RSS wall applications?	This part of Section 2 relates to implications for the overall design, geometry, and roadside safety required when using RSS walls, not to the design of the RSS walls properly. Therefore, it will remain in Section 2.5.1. Submerged RSS wall application is not covered by the MTO DSM and is not considered a standard design. If required for a project the design provision shall be determined at the project level.
395	**Individual**	Section 5.5: (b) It's a bit confused if approach slab will be longer than wingwall, why length of wingwall needs to be extended. (c) SS 105-15,16,17 could not be found in Technical Publication Site.	b) We have corrected the requirement. The approach slab should be extended to terminate beyond the end of the wingwall. c) SS105-15,16 &17 drawings will be available by end of spring 2024.
396	**Individual**	Section 8.1.2 last sentence of bullet point (b) as well as bullet point (c) is redundant. As all members are AT or WT grade according to bullet point (a) Bullet point (d) allows non notch-tough steel which is in conflict of bullet point (a) or this is supposed to be an exception. If not, what the notch-toughness requirement for secondary members in general, as Bullet point (e) describe the notch-toughness requirements for Secondary members of curved bridges or highly skewed bridges only.	We do not see a conflict. AT or WT notch toughness of rolled sections may not always be available and c) permits A or W for specific cases.

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Comments received by TCP (000-0184)			
397	Individual	section 1, page 5 of 16, 4.4.3.1 title was repeated, need to delete the latter one. page 6 of 16, section 5.5.3, allow more of the voids seems have little use. Since void slab rarely has deep section. section 6.9 this should be consulted with MTO foundations. section 2, page 13 of 46, section 2.4.5, paragraph 1, maybe non-reduction only apply to overpasses. when major highway with high speed underneath, the 10mm extra cover definitely helps. section 5, page 11 of 15, could just use MSE, to avoid repeating. it is easy to understand. using interchangeable is wasting typing efforts.	Section D1S1-4.4.3.1: corrected Exceptions to Section 5.5.3: Is based on MTO's historical practice Section 6.9 (Exceptions): This clause allows for MTO Policy memo 2020-01 (March 2020), which was developed for MTO projects. Section 2.4.5: the non-reduction is long-standing MTO policy of having 40mm cover to thin deck slabs and 50mm for thick slabs. Section 5.3 and 5.4: MTO is switching from RSS to MSE, however numerous documents still contain the RSS term and keeping both terms will continue until the terminology changes through all documents.
398	Individual	Section 8.3 Last sentence of the second paragraph: For example, a girder bottom flange should not be oriented level in the transverse direction, even if the web needs to be out of plumb. Will this affect the geometry of shoe plate? What's the recommended slope for bottom flange orientation in the transverse direction then?	The section has been revised.
399(a)	Individual	There is a discrepancy between Figure 8.1.1 and the corresponding text under Section 8.1.2 n) where 2mm is shown in the figure while the corresponding text states 1.5mm section loss to be assumed.	The figure has been updated.
400	Individual	I think is just a typo on Section 1 in 8.12.3.4 db should be in mm and not mm2. I remember using the equation before and the same typo was present. Thanks!	Thank you. Corrected.

Comments received by TCP (000-0184)			
402	**Individual/Morrison Hershfield**	I had to submit my comments this way because I got an error message trying to log in: "Unable to send email. Contact the site administrator if the problem persists.: Comments: Section 1 8.8.4.6 (ii) An existing typo has not been corrected: “For all prestresses concrete elements, the limiting concrete tensile stress at transfer shall "be?" 0.6fcri.” Section 8 8.1.2(n) and Figure 8.1.1 It is not clear if loss of steel section shall be assumed as 1.5 mm or 2 mm. Section 16 16.5.1 There is a possible typo: "The maintenance vehicle is specified in CSA S7". Should it be CSA S6?	Thank you, corrections are made. The maintenance vehicle refernce to CSA -S7 is correct

Ontario

<table>
<tr><th colspan="4">Comments received by Email</th></tr>
<tr><th>Comment Number</th><th>Organization</th><th>Comment</th><th>Response</th></tr>
<tr><td></td><td></td><td></td><td></td></tr>
<tr><td>1</td><td>EMO/MTO</td><td>
<table>
<tr><th>Section</th><th>Comment</th></tr>
<tr><td>Section 7.2.1:
Note: The availability of GU ~~and HE~~ cement will be severely restricted in late 2024 as the cement industry in Canada moves exclusively towards GUL Portland-limestone cements to reduce environmental impacts. Therefore, in the near future it will no longer be possible to achieve 45 MPa transfer strength with a single day turnaround.</td><td>

Almost all plants producing GU in Ontario have now switched to GUL
GUL is not replacing HE. HEL will be placing HEL but this process is too slow because of the resistance from the precast industry
I am not sure why the 45 MPa can not be achieved anymore? HEL will be produced to be equivalent to HE in terms of strength the same way GUL is being produced equivalent to GU.
</td></tr>
<tr><td>Section 5.2.1.4
Concrete with a shrinkage compensating admixture should be……</td><td>Concrete with a shrinkage reducing or compensating admixture should be……</td></tr>
</table>
</td><td>In Section 7.2.1, HEL has been added as well as an explanation for avoiding transfer strenghts of 45 Mpa or higher.

After further discussion, the reference to concrete with shrinkage reducing admixture has been removed.</td></tr>
</table>

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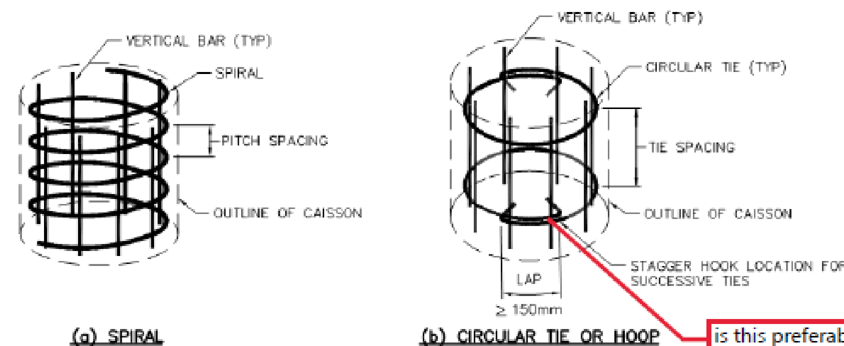
Comments received by Email			
Comment Number	Organization	Comment	Response
2-1	**West Region**	**Section 3.3.1** Spiral pitch/hoop This may be problematic, especially for long caissons with temporary liners. DOT did some research with the University of South Florida and found concrete builds up a head differential inside the rebar cage before finding its way through the rebar. They found there was a relationship between CSD (the ratio of the clear rebar spacing and aggregate size) and the head differential that builds up inside the cage. They were measuring head differentials in the magnitude of feet. A head differential like this could cause a cage to collapse during liner extraction. This is my leading theory for a number of very costly caisson cage collapses that have happened recently in WR. See FDOT paper "Factors Affecting Anomaly Formation in Drilled Shafts" Mullins et all 2005 Currently the AASHTO design code states the clear spacing between bars must be at least (clear between bars, not bar spacing): • Five times the maximum aggregate size (95mm in our case) • 5.0 in. FHWA drilled shafts manual recommends a minimum spacing of 8 times the aggregate size (152mm in our case), and indicates some agencies require a spacing of 10x. (190mm) Transverse hoop reinforcement is not required to be this tight by S6 for compression members.	The current design requirements for spirals in compression members in both CSA-S6 and AASHTO is technically the same approach using a minimum volumetric reinforcement ratio provided in the section. MTO has been working extensively with geotechnical engineers, deep foundation contractors, rebar fabricators/suppliers and concrete material specialists to develop a standard NSSP for drilled shaft foundations which is implemented and will be published as an SP in the near future. Based on the outcome of new development (i.e., 500W steel adoption, use of 13 mm max coarse aggregates, practical pitch spaces and design requirements, 20M spiral availability, and alternative hoop details), the proposed transverse reinforcement design is recommended to meet the code design and readily achievable details on site in Ontario.

Ontario

Comments received by Email			
Comment Number	**Organization**	**Comment**	**Response**
2-2	**West Region**	not entirely accurate, there is a zone of contaminated concrete, typically construction practices allow it to be mostly controlled ot the top of the pour... RUCTURAL MANUAL DIVISION 2 - PROCEDURES SECTION 3 - DEEP FOUNDATIONS 3.1.3 Concrete Placement The method used to place the concrete falls into one of two categories, wet pour, or dry pour. The location of the water table and the type of materials that make up the underlying geology control whether the excavation is considered a "wet" shaft or a "dry" shaft. For wet shafts, the concrete must be placed through a tremie or pump extension so that the water does not mix with the concrete as it is being placed in the excavation. A wet pour requires the use of a permanent casing. casing can be temporary 3.3.1.4 Rebar Cages	A general statement for wet tremie pour explained and the section is updated.
2-3	**West Region**	3.3.1.4 Rebar Cages A drilled shaft rebar cage is comprised of longitudinal bars that are normally arr... a uniform spacing circumferentially to form a cylinder that is concentric with th... shaft. Transverse reinforcing is placed around and attached to the longitudinal b... commentary on minimum clear spacing, at least for long caissons with temporary liners, should be included. The most common types of transverse reinforcement in drilled shafts are spirals. 15M spirals are typically used for caisson piles and readily available from rebar fabricators. The tight pitch spacing on spiral reinforcement can often result in constructability issues with concrete flow through the rebar cage. When a design pitch spacing of 15M spirals is less than 80 mm, use of circular ties or hoops with either same bar size and/or larger bar size can allow an increase in the bar spacing as shown in Figure 3.3.1. Another solution which can be designed by 20M spirals with spacing 50% larger than 15M spirals. The designer shall confirm the availability of 20M spiral reinforcement from the rebar fabricators. In any case, the spiral pitch cannot exceed 6 times the diameter of the longitudinal reinforcement, nor 150mm. Mixing of spirals and hoops within the same section is not permitted.	The section has been updated to address this.

Comments received by Email			
Comment Number	**Organization**	**Comment**	**Response**
2-4	**West Region**	 **Figure 3.3.1 – Transverse Reinforcing Details for Caisson Pile** is this preferable to welded hoops? is full lap length required?	Yes, this is a preferable detail based on consultations with rebar fabricators because welded hoop are not practical. The detail is also recommended by ACI 318-14. Section has been updated to be clear that the lap shall be greater than 150 mm but need not be a full lap splice.
2-5	**West Region**	3.3.3 Drawings Standards notes for drilled shafts shall be listed on the drawings. The following are typical drilled shaft notes for various conditions. 1. CAISSONS ARE .... Mm NORMINAL DIAMETER AS SHOWN AND SHALL BE DRILLED AND SOCKETED INTO BEDROCK. 2. MAXMUM COMBINED FACTORED LOADS: SLS .... kN PER CAISSON ULS .... kN PER CAISSON STEEL CASINGS FOR CAISSONS SHALL CONFORM TO ASTM A252 GRADE 3 MODIFIED (345 MPa). SEAMS OF WELDED CASING SHALL BE SEAMLESS HELICAL BUTT AND COMFORM WITH THE REQUIREMENTS OF CSA W59. above it says that longitudinal welds are acceptable	The note is updated.

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Comments received by Email			
Comment Number	**Organization**	**Comment**	**Response**
2-6	**West Region**	**3.4 Pile Caps** The pile cap is a reinforced concrete slab or block which interconnects a group of piles and acts as a medium to transmit the load from wall or column to the piles. The pile cap shall be rigid so as to distribute the forces equally on the piles of a group. In general, it is designed like a footing on soil but with the difference that instead of uniform reaction from the soil, the reactions in this case are concentrated either point loads or distributed. The thickness of the pile cap is typically established to resist shear without need for shear reinforcement and should be sufficient for the bars projecting from the piles and the dowel bars for the columns to be developed. Where a pile cap meets the definition of a deep beam according to the CHBDC, the pile cap shall be designed using a strut-and-tie model. i don't believe this is ever strictly true. we typically have some eccentricity or lateral load. also it's never truly rigid, though in many cases we assume so. I'm recalling a structure with very wide pile spacing. the thickness of the pile cap would be assumed rigid in normal cases, but in this case the flexibility was not negligible.	The section has been revised.
2-7	**West Region**	**5.1.1 Integral Abutments** Integral abutment bridges are single span or multi-span bridges with a movement system composed primarily of abutments on flexible integral pile foundations and approach slabs, in lieu of movable deck expansion joints and bearings at abutments. The effect of the longitudinal forces in the structure due to temperature, shrinkage and creep is minimised by making the abutment foundations flexible and less resistant to longitudinal movem Integral abutment bridges are well-suited for the concrete slab-on-girder typ superstructures for total bridge length of 150 m with thermal movements of a maximu 75 m length going to one side, and skew angle of less than or equal to 20 degrees. To minimise the effect of soil pressure and resistance to abutment movements, the total height of the abutment wall and length of wingwall should not exceed 6 m and 7 m respectively. The abutment should be supported on relatively flexible piles such as H-piles. Where the load-bearing strata is near the surface or where the use of short piles less than 5 m in length is planned, the site may not be suitable for integral abutment bridges. Integral abutments may be constructed on concrete filled-tube piles, concrete caissons, or on columns supported on spread footings (e.g., spill-through abutments) where these systems can provide the flexibility needed to accommodate the movements from the superstructure. consider providing guidance on orientation of H	This is covered by the existing integral abutment report, and that guidance will be updated in the future.

Comments received by Email			
Comment Number	Organization	Comment	Response
2-8	**West Region**	the abutment wall. **5.2.1.4 Abutment Wall Vertical Construction Joint** Relatively long and thick abutment walls are prone to vertical cracking due to res[…] shrinkage and thermal effects. For abutment walls placed on spread footings or p[…] and when the length of the abutment wall exceeds 12 m, vertical construction joints shall be specified to construct the wall in shorter lengths to control cracking due to restrained shrinkage. Smaller diameter reinforcing steel at tighter spacing is more effective to control cracking than equivalent quantity of steel with larger bars at larger spacing. The reinforcing steel at the front face of the abutment shall have a spacing of not more than 150 mm. Concrete with a shrinkage compensating admixture should be specified for thick walls. consider specifying thickness. How does this relate to the requirement above for using corbels?	Section has been updated.
2-9	**West Region**	**Section 5.2.2** STRUCTURAL MA[…] DIVISION 2 - PR[…] SECTION 5 - ABUTMENTS, WI[…]TAINING WAL[…] this is the opposite direction to figure 5.2.1 below, is this intentional? should this say that it should follow the crossfall? with wide bridges a horizontal bearing seat entails some very high pedestals. this isn't clear, what is an "open joint?" Does this mean the bearing seat should be 1 vertical to 3 horizontal? the front face at 5% min. under a sealed joint and at 1 in 3 min. under an open joint. In the direction parallel to the front face, this surface should be horizontal for simplicity. Structural steel bridges require special treatment to prevent rust staining of piers and abutments. For a standard detail on piers, see Section 6.2.3. For a standard detail on abutments, see Figure 5.2.1.	Section has been updated.
2-10	**West Region**	STRUCTURAL MANUAL DIVISION 2 - PROCEDURES **SECTION 6 - PIERS** **6 PIERS** **6.1 Design** there can be advantages to using elastomeric bearings at the pier in zones requiring seismic considerations. Preference is to provide integral piers for prestressed girder bridges in lieu of shallow bearings. These bridges require cast-in-place concrete in the pier, so this adds no additional construction operations, it improves structural redundancy and reduces the number of elements requiring future inspection and maintenance. For more than 2 spans, multiple integral piers are possible, however the piers must be able to accommodate the superstructure thermal movements.	The statement does not preclude the use of bearings, but states a clear preference to avoid them.

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Comments received by Email			
Comment Number	**Organization**	**Comment**	**Response**
2-11	**West Region**	STRUCTURAL MANUAL DIVISION 2 - PROCEDURES SECTION 6 - PIERS this is repeated below in e). requirements for crash protection walls, and for crash load. These vary by railway owner and must be confirmed during bridge design. b) New Bridge piers with only one or two columns shall be designed for the CHBDC Collision load regardless of their distance from the edge of the travelled lane.	The repeated statement has been deleted.
2-12	**West Region**	**Ref: Section 6.1.4c and last Paragraph:** Can we decide on a uniform policy for the treatment of existing piers? Individual structural sections are not really equipped to make this call in each instance. In WR, based on previous consultation with Bridgecom, we've been generally upgrading piers when a superstructure replacement is completed, but not upgrading when less of a rehabilitation is completed. Can this become policy? We could also perhaps have an ADTT cut-off specified. Based on their remaining service life, it seems that it would often be a poor use of resources to strengthen columns built before the 79 code.	This policy may be adjusted in the future.
2-13	**West Region**	**6.2 Miscellaneous Details** **6.2.1 Pier Nosing for River Piers** Steel nosing should not be provided unless specifically called for in the Structural Design Report or in subsequent correspondence by the Head of Structural Section. can we add some clarity for those preparing our SDRs? Can we clarify that steel nosing shouldn't be used for rivers such as Credit River or Grand River? Is this the intention? January 2024 S06 DRAFT	This section has been updated to explain that steel nosing applies to angular piers in rivers with heavy ice floes.

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Comments received by Email			
Comment Number	Organization	Comment	Response
2-14	**West Region**	STRUCTURAL MANUAL DIVISION 2 - PROCEDURES SECTION 6 - PIERS **6.2.2 Pier Bearing Seats** If the bridge deck over a pier is not continuous, the requirements of Section 5.2.2 concerning the sloping of the bearing seat ledge apply also to the pier. can we clarify? which direction should the slope be? The requirements of Section 5.2.2 concerning provisions for jacking apply also to **6.2.3 Rust Stain Control for Steel Girder Bridges** Piers are to be detailed so that bearing seats are above the dams. The slope match the deck crossfall. Add a counter slope to direct staining away from the en slope should be of equal magnitude (max. 4%) but opposite direction to the dec fall. consider clarifying. Perhaps "piers are to be detailed with rust dams below and surrounding the bearing seats to carry rust contaminated water towards a vertical drain groove."	This section has been updated.
2-15	**West Region**	Precasters are reluctant to stress the strands above 0.80 f_{pu} due to the breakage. This risk is higher for deflected strands. Consequently, the design limit the specified prestressing stress prior to transfer to 0.74 f_{pu}. current drawing on CPS is for CPCI girders, will this be updated? i) Details of the positive moment connection over piers are given on OPSD 3310.150. In integral and semi-integral abutment situations, the connection of the girder to the abutment, is achieved by the use of projected L-shaped reinforcement bars or by bent projected strands from the girder. j) All diaphragms shall be cast integrally with the deck slab pour, without construction joints. Diaphragms shall completely encase strands at the ends of girde the minimum cover required by the CHBDC. k) The girders' 'stirrups' projection above the top of girder must be ca specified so that it accounts for girder hogging (upward deflection) an curve of the roadway profile. It shall be verified and revised, if necessa that the girder stirrups are projecting a minimum of 25 mm above reinforcement mat of the deck slab. The stirrups at the girder ends minimum spacing of 75 mm to avoid reinforcement congestion. Partic shall be given to the spacing and arrangement of stirrups at the vicinity of the rectangular dowel holes. In the case of skewed bridges, a plan detailing the arrangement of the stirrups and the dowel holes shall be included on the drawings. 25 clear space or centre-centre? consider clarifying if hat-bars are acceptable, and if so, if the top legs should have lap lengths or standard hook lengths.	i) is corrected to reference the SSDs instead of OPSD.

<table>
<tr><th colspan="4">Comments received by Email</th></tr>
<tr><th>Comment Number</th><th>Organization</th><th>Comment</th><th>Response</th></tr>
<tr><td>2-16</td><td>West Region</td><td>+G+D-C

The purpose of the bearing soffit undercut is to ensure proper contact between the girder and the elastomeric bearing when all the dead loads have been applied. In calculating the undercut, the structure grade ‘G’, camber due to prestress ‘C’ as well as deflections due to the girder self weight and the wet concrete ‘D’ should be considered (camber and deflections in this case refer to the resulting rotations). At the low end, dimension ‘b’ shown on the Structural Standard Drawing SS 107-X is a function of +G-D+C. At the high end, dimension ‘c’ shown on the Structural Standard Drawing SS 107-X is a function of -G-D+C. Therefore ‘b’ and ‘c’ may differ.</td><td>The calculation has been reviewed and updated. "At the low end, dimension ‘b’ shown on the Structural Standard Drawing containing the girder details is a function of +G-D+C. At the high end, dimension ‘c’ is a function of +G+D-C." A diagram will be added to the forthcoming Prestressed Concrete Girder Guidelines to make it more clear.</td></tr>
<tr><td>2-17</td><td>West Region</td><td>7.3.1 General

a) Post-tensioned superstructures, which are solid or voided by means of round tubes, must be transversely prestressed throughout their length with reinforcing steel reduced to a minimum. Transverse stressing is not mandatory for box section decks except as required by (c) and (d) below.

b) For skew angles in excess of 20º, transverse prestressing cables and ~~reinforcement~~ reinforcing steel should be square to the deck except over skewed supports.

c) Transverse moments over piers and abutments shall be resisted by tra prestressing rather than reinforcing steel.

d) Wherever possible, the cantilever portion of cast-in-place, post-tensioned section shall be greater than 1.6 m. Deck cantilever overhangs exceeding 3 length shall be prestressed.

does this mean that all the transverse PT should be skewed if supports are skewed, or that PT and/or reinforcing should be flared?</td><td>The transition could be flared or short tendons used, depending on the width of the bridge and skew angle.</td></tr>
<tr><td>2-18</td><td>West Region</td><td>7.3.6 Post-Tensioning Tendons and Duct Sizes

Design of post-tensioning shall be done with commonly stocked tendon sizes, and standard plastic duct sizes as shown in Table 7.3.1. Duct sizes are established to ensure the inside cross-sectional area of the duct is at least 2.5 times the net area of the strand. Duct diameters given are nominal and actual diameters can vary by ± 3 mm.

consider clarifying when steel should be used

Table 7.3.1
<table>
<tr><th>No. of 15 mm Strands</th><th>Plastic Dust I.D./O.D.</th><th>Steel Duct I.D./O.D. (mm)</th></tr>
<tr><td>5</td><td>48/59</td><td>55/60</td></tr>
<tr><td>7</td><td>59/73</td><td>65/70</td></tr>
<tr><td>12</td><td>76/91</td><td>85/90</td></tr>
<tr><td>19</td><td>100/116</td><td>105/110</td></tr>
<tr><td>27</td><td>115/135</td><td>125/130</td></tr>
<tr><td>37</td><td>130/151</td><td>135/140</td></tr>
</table></td><td>Added an explanation that steel ducts dimensions are provided for reference only.</td></tr>
</table>

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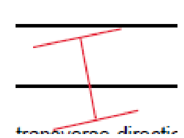
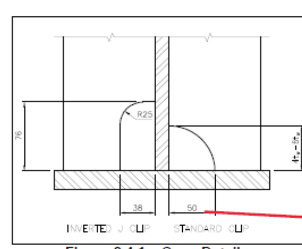
Comments received by Email			
Comment Number	**Organization**	**Comment**	**Response**
2-19	**West Region**	Section 8.1.1 water or vegetation growing against the bridge; • Roadway or marine salts that slow the drying process and accelerate corrosion, effectively reducing relative humidity to 60%; and this statement isn't clear • Debris that traps moisture.	The clause has been revised and the reference to relative humidity has been removed.
2-20	**West Region**	**Section 8.1.2 Structural Steel Design Requirements:** Figure 8.1.1 indicates 2mm Loss of steel while note "n" says 1.5mm which number is correct?	The figure has been updated.
2-21	**West Region**	**8.1.9 Structural Steel Box Girders Temporary Bracing** For concrete deck slabs on steel girder bridges to be designed using the empirical method, the CHBDC requires that cross frames or diaphragms, at a maximum spacing of 8.0 m c/c, be provided throughout the full cross section width of the bridge, inside and between box girders. When such diaphragms or cross frames are not provided, temporary bracing to prevent displacement or twisting of the girders may be required, particularly when the deck is placed. The designer shall check the stability of the girders during the deck, and if placement	The sentence has been corrected.
2-22	**West Region**	**Section 8.3 Details: In the last part of the second paragraph It is not clear what is meant by "out of plumb" is this sketch the intention? Isn't one side even worse than level?** 	The sentence has been deleted.

Ontario

Comments received by Email			
Comment Number	Organization	Comment	Response
2-23	**West Region**	**Section 8.3 Details:** In the last paragraph all of these are often necessary details. instead of "avoid" should this say "minimize?" Also, is the intention to minimize these at any cost? or within some reasonable parameters? we could eliminate a lot of stiffeners by using really thick web plates, is this the intention? we could eliminate bolted splices with field welding?	The requirement to avoid does not mean these details are prohibited. In the next update, we will consider if we can better define the expectation and balance of detailing.
2-24	**West Region**	**Section 8.3.1 Structural Steel Box Girder Bottom Flanges:** In last paragraph & second line should it be weld or Caulk?	We have revised this clause to avoid sealing copes with welds.
2-25	**West Region**	8.3.2 Box Girder Drainage and Ventilation Drains are required through the bottom flanges of box girders wherever water c[...] These should be detailed to prevent water from running along the soffit and t[...] birds. As a minimum, drains should be located at each end of every span [...] spots. Drains shall project 50 mm below the bottom of the bottom flange. [...] longitudinal stiffeners shall be detailed at drains as necessary. Drains shall be located to avoid staining of the substructure. at integral substructures this is difficult, unless drainage tubes are extended.	We will consider a more prescriptive requirement for the next update to the Structural Manual.
2-26	**West Region**	8.4 Stiffeners and Connection Plates be 8.4.1 Coping of Stiffeners and Gusset Plates Copes on transverse stiffeners shall inverted J-clips, with at least the minimum dimensions shown in Figure 8.4.1. Copes on other details, such as longitudinal stiffeners and gusset plates, shall be quarter-round, not less than 50mm in radius (see Figure 8.4.2).  50 min? Figure 8.4.1 – Cope Details These larger copes are desirable for the following reasons: 1) They prevent the possibility of intersecting welds; 2) They reduce the high weld shrinkage strains associated with smaller copes; and, 3) They allow drainage without the buildup of debris. mesh and caulking is specified above. At end diaphragms of box girders, copes shall be filled with weld or caulking to prevent entry of rodents or birds. This generally dictates a drain at the diaphragm.	The figure has been updated.

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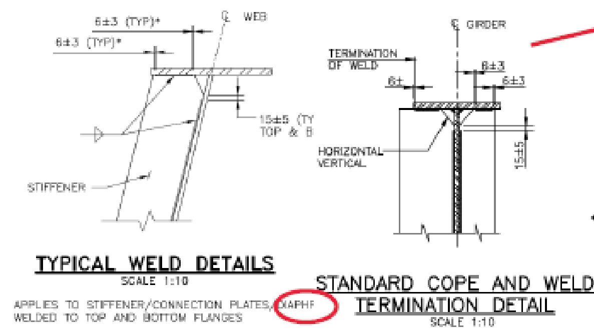
Comments received by Email			
Comment Number	**Organization**	**Comment**	**Response**
2-27	**West Region**	Fillet weld hold backs shall be indicated on the contract documents.  Figure 8.4.3 – Weld Termination Details suggest this figure be updated to show J clips.	The figure has been updated.
2-28	**West Region**	**8.4.3 Lateral Bracing** Lateral bracing shall be provided only where required and shall be connected directly to the flange where feasible. When it is not feasible, lateral bracing may be connected to lateral gusset plates. All gusset plates for lateral bracing should be fillet welded and be located a distance as required by the CHBDC and practical situations. The outer corners of the gusset plates should be left square unless fatigue design requires a radiused gusset plate. "Bridge Fatigue Guide, Design and Details" by J. W. Fisher should be consulted when determining the location of bolt holes. See also Figure 8.4.2. Several factors should be taken into consideration in [...] bracing gusset plates. 1) Access for fabricating and inspecting the gusset pl[...] 2) The fatigue performance; lateral bracing bolted directly to the flanges has superior fatigue performance, whereas gusset plates can be moved away from the flange into a lower stress region. For girders with a depth of up to 2.4 m, the bracing shall be connected to the top flange or connected to gusset plates installed close to the top flange. this makes formwork difficult, is this justified when substantial diaphragms are used? The requirement only for shorter girders isn't clear, can this be clarified?	The experience to date indicates that the long-term benefits outweigh the short-term challenges with forming the deck locally over lateral bracing. Lateral bracing can and should be detailed over only a portion of the span where it is required.

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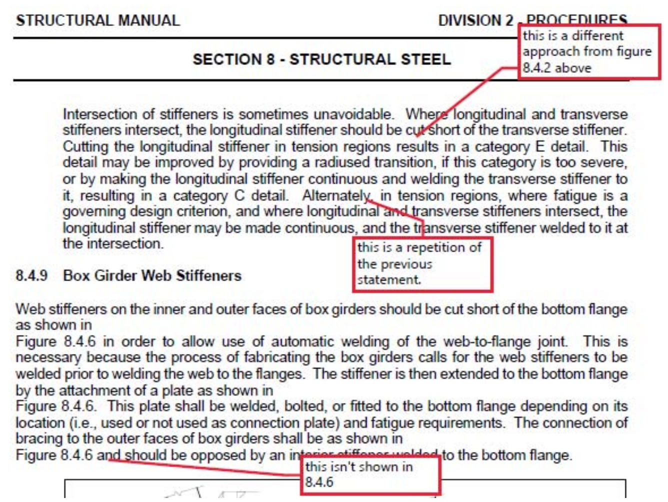
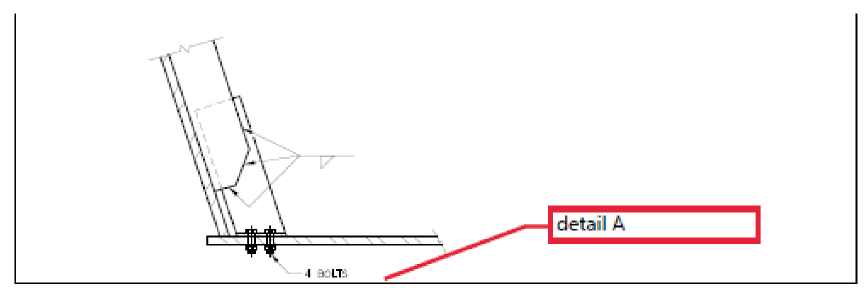
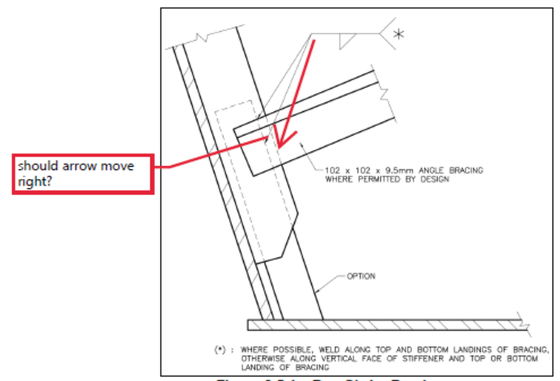
Comments received by Email			
Comment Number	Organization	Comment	Response
2-29	**West Region**	 STRUCTURAL MANUAL DIVISION 2 - PROCEDURES SECTION 8 - STRUCTURAL STEEL Intersection of stiffeners is sometimes unavoidable. Where longitudinal and transverse stiffeners intersect, the longitudinal stiffener should be cut short of the transverse stiffener. Cutting the longitudinal stiffener in tension regions results in a category E detail. This detail may be improved by providing a radiused transition, if this category is too severe, or by making the longitudinal stiffener continuous and welding the transverse stiffener to it, resulting in a category C detail. Alternately, in tension regions, where fatigue is a governing design criterion, and where longitudinal and transverse stiffeners intersect, the longitudinal stiffener may be made continuous, and the transverse stiffener welded to it at the intersection. this is a different approach from figure 8.4.2 above this is a repetition of the previous statement. 8.4.9 Box Girder Web Stiffeners Web stiffeners on the inner and outer faces of box girders should be cut short of the bottom flange as shown in Figure 8.4.6 in order to allow use of automatic welding of the web-to-flange joint. This is necessary because the process of fabricating the box girders calls for the web stiffeners to be welded prior to welding the web to the flanges. The stiffener is then extended to the bottom flange by the attachment of a plate as shown in Figure 8.4.6. This plate shall be welded, bolted, or fitted to the bottom flange depending on its location (i.e., used or not used as connection plate) and fatigue requirements. The connection of bracing to the outer faces of box girders shall be as shown in Figure 8.4.6 and should be opposed by an interior stiffener welded to the bottom flange. this isn't shown in 8.4.6	The figure has been updated.
2-30	**West Region**	 January 2024 Page 24 of 37 SM-D2-S08 DRAFT	The figure has been updated.

Comments received by Email			
Comment Number	Organization	Comment	Response
2-31	**West Region**	 Figure 8.5.1 – Box Girder Bracing	The figure has been updated. The arrows were shown incorrectly.
2-32	**West Region**	**8.7 Structural Steel Notes** 16. If the bridge is a multi-span steel box-girder structure, the following note 15 should be included: ADJUSTMENTS SHALL BE MADE TO THE RELAXED CAMBER DIAGRAM TO COMPENSATE FOR THE DEFLECTION OF THE INDIVIDUAL GIRDER SEGMENTS. consider clarifying that this applies only to I girders and 17. The designer shall add the following note to the structural steel drawings at exterior girder field splice locations, unless the entire exterior I-girder is coated: ALL STRUCTURAL STEEL SURFACES OF EXTERIOR I-GIRDERS, INCLUDING SPLICE PLATES, BUT EXCLUDING SURFACES IN CONTACT WITH CONCRETE AND THE CONTACT SURFACES OF BOLTS JOINTS, SHALL BE COATED FOR A DISTANCE OF 2000 mm ON EITHER SIDE OF THE CENTRELINE OF A FIELD SPLICE.	The note has been revised to make it clear that it is only for I-girders.
2-33	**West Region**	a) Typical Haunches b) Deep and Narrow Haunches **Figure 9.3.2 – Haunch Detailing** S6 actually calls for 25mm clear from underside of head to top of transverse steel haunch Shear stud height in a steel girder or stirrup projection in a concrete girder shall extend a minimum of 25 mm above the bottom mat of bars. Haunches shall not be reinforced unless the haunch depth above the flange exceeds 100 mm. Stirrup projections and shear stud height shall be designed as necessary to avoid additional flange reinforcement except when stirrup projections or shear stud length exceeds 300 mm, in which case haunches shall be reinforced to extend the bottom mat of reinforcing steel downwards into the haunch.	This section has been updated to reflect the code's requirements for both shear studs and stirrups.

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Comments received by Email			
Comment Number	Organization	Comment	Response
2-34	**West Region**	Typically, steel girders are fabricated to follow the roadway profile through built-in camber and a uniform haunch thickness is achieved along the girder length and in transverse direction, whereas concrete girders require a variable haunch to make up the difference between the highway profile and the deformed shape of the girder prior to casting the deck. Nevertheless, the actual haunch on site could vary from estimated may affect the stirrup projection in the deck. When stirrup projection turns o 25 mm above the bottom mat of deck reinforcement, the haunch is r reinforced with transverse bars, usually in this shape to interlock stirrups or studs. Where stirrup projections or shear studs are too long and impede cover to the top of deck, consideration can be given to bending them to achieve the cover. what length of horizontal leg is required? development, lap, or standard hook?	The sentence has been updated to clarify that standard hooks are sufficient.
2-35	**West Region**	The deck placing sequence should be shown in numerical order. NOTE: Simultaneous concrete placements should not be specified u necessary, in which case the intent should be clarified on the deck slab 9.4.3 **Screed Elevations on Bridge Decks** Screed elevations are the elevation to which the deck needs to be plac final vertical profile after all dead load deflections occur. Screed elevati by the contractor by adjusting the height of the haunches as required. 9.4.3.1 **Slab on Girder Decks** Screed elevations shall be given at the centreline of all exterior girders, the break points in the deck, and on the deck at the faces of curbs and barrier walls. Screed elevations shall be given at intervals not exceeding 3 m. Screed elevations should include an allowance for long term dead load deflection. Seems to contradict 7.2.7 "Screed elevations shall not be established using long-term deflections.…The deflection due to the weight of the wet concrete slab and superimposed dead load shall be multiplied by a factor of 1.10"	This sentence is deleted.

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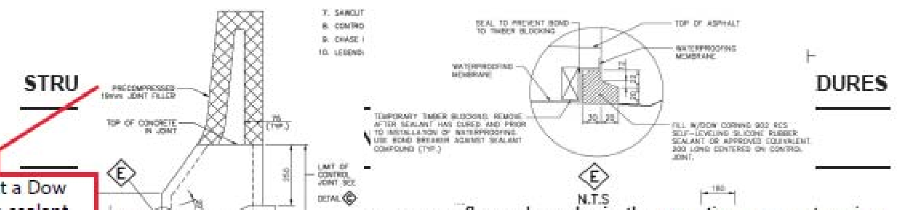
Comments received by Email			
Comment Number	**Organization**	**Comment**	**Response**
2-36	**West Region**	 What about a Dow coring type sealant along the joint? numerous flexural cracks in the negative moment region. ...oofing system shall be according to ... sed. The concrete sidewalk shall b... waterproofing membrane, a minimum of 1 day after the waterproo... is the underlying concrete intended to be smooth? This waterproofing should start at 150 mm in front of the face of the sidewalk, continue along the top of the deck/sidewalk construction joint and be turned downwards along the vertical joint face under the barrier wall. The waterproofing system should extend the full ...ngth of the bridge. a figure would be very helpful. we're going to get quite a mess dumping waterproofing down the vertical face. ...henever this requirement is used, additional dowels should ... he ...ck/sidewalk interface to ensure continuity and the integrity of the deck cantilever to ...sist the traffic barrier loads given in Clause 3.8.8.1 of the CHBDC. what size, how many? Our designers don't know how to detail this. 9.5.4 Structure Deck Drainage	The section has been updated to clarify these items. A figure will be added to a future version of the manual.
2-37	**West Region**	10.5 Barrier Walls Beyond the Bridge Structure 10.5.1 Barrier Walls in Fill Piles a figure would add a lot of clarity here. I'm not sure what this section is about. The length of piles for barrier walls on fill shall be determined as follows: a) Piles located between the structure and first pavement expansion joint from the structure: Piles 1 m into existing ground, or minimum overall length 3 m and maximum overall length 6 m;	The section has been reworded to make the intent clear. We agree, the need for barriers on piles is uncommon.
2-38	**West Region**	**10.6.2 Inspector Guards** 2. The guard shall be mounted on the wall, within 300 mm of the exterior face of the wall. 3. Posts shall be installed vertically. this isn't clear 4. Posts shall be mounted to the retaining wall with base plates and anchors designed to resist the loads imposed on the guard. Anchors shall be embedded into the retaining wall or anchored with epoxy. Given their history of problematic maintenance, posts shall not be embedded directly into the concrete, nor shall anchors be used to affix base plates.	This section has been updated for more clarification.

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Comments received by Email			
Comment Number	Organization	Comment	Response
2-39	**West Region**	are **11 RIGID FRAMES** **11.1 General** Rigid frame structures ideal for short to medium span bridges. The high degree of structural indeterminacy allows redistribution of forces between the deck and the substructure contributing to resilience in the face of extreme events. The jointless nature of a rigid frame structure offers a sustainable structure with low maintenance efforts during the service life of the bridge.	Thank you, this has been corrected.
2-40	**West Region**	**12.2 Premium Reinforcing -Where Required** Even for bridge decks that are waterproofed, those on busy highways have additional wear of the waterproofing due to heavy traffic while simultaneously tending to have rehabilitation and waterproofing replacement delayed due to the desire to avoid traffic disruptions. For these bridges, there are benefits to using Premium Reinforcement in the deck. Table 12.2.1 identifies bridge decks that require Premium Reinforcement. suggest you explicitly state here that all top reinforcement in high AADT decks must be premium. For bridge rehabilitation: • 2304 Duplex shall be specified. It is expected that Type 2304, with its lower content will be less expensive, but still adequate for rehabilitation service life;	For Deck Top and closure pores, Table 12.2.1 specifies the AADT>50,000.
2-41	**West Region**	**12.2 Premium Reinforcing -Where Required** I don't think we want to replace members, ie bridge decks, pier columns, and barriers, for no other reason than that they don't meet the premium reinforcement requirements at the first rehab. I think the previous bullet should apply to all bridges with remaining life beyond 35 years. We have lots of bridges with epoxy coated bars in the barriers and pier columns with more than 45 years of expected service life remaining (most bridges built in the 90's). I don't think we want to fully replace all these barriers at the upcoming rehab.	The bullet has been modified to clarify that it is intended to apply only to components which require replacement, not to trigger replacement.

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Comment Number	Organization	Comment	Response
2-42	**West Region**	Table 12.2.1 – Reinforcing Requirements for Surfaces within Splash Zone Component Surface / Reinforcement[6] Deck Top: Deck Closure pours between Precast components. – Stainless or GFRP Deck Top within 1.5 m of expansion joint gap. – Stainless Topping Slab within 1.5 m of expansion joint. – Stainless Deck Top and closure pours between CIP deck stages. – Stainless or GFRP on freeways with AADT > 50,000 Sidewalks and Barriers: Barrier and parapet walls. – Stainless or GFRP[1] Sidewalks, medians, and curbs. – Stainless or GFRP Deck Soffit: See Figure 12.2.1, Figure 12.2.2 and Figure 12.2.3. – Stainless or GFRP Deck soffit within 1.5 m of expansion joint gap. – Stainless Soffit of Post-tensioned Bridges with AADT > 50,000 under bridge. – Stainless or GFRP Precast deck. – Same as CIP deck. Girders: Stirrups and perimeter bars from precast component (i.e., NU, CPCI, box) within 1.5 m of expansion joints. – Stainless[2] Front surface of ballast wall and top surfaces of bearing seats and pedestals exposed to roadway drainage or possible dripping[3]. – Stainless[2] Surfaces of abutments, wingwalls, retaining and ... seems to be an inconsistency here between PT decks and Precast girders which could unintentionally discourage the PT option during pre-design. Suggest additional clarity be added to make it clear that deck top steel must be premium for high AADT structures. Suggest adding clear language above stating that all the deck top steel in high AADT structures must be premium. It would help to provide a separate row in this table for deck top reinforcement. When I first read this table, it looked like the "and" was a typo, and that the intention was that premium was only required "at" the closure pour. This is a major policy change from versions before 2021 and I feel we should make it very clear that it is intentional.	The deck top requirement has been split into a separate row. A note has been added to clarify the requirement for PT bridges and to explain that they have a longer design service life.
2-43	**West Region**	assumed to develop 100 kN of bar strength. **12.5 Anchors in Concrete** Anchors post-installed into concrete shall be adhesive ... grouted bonded and screw types shall not be used. **12.5.1 Post-installed Adhesive Dowels in Concrete** Dowels into concrete shall not be used in new structures, except for reinforcement installed through steel or precast concrete girders at integral abutments and piers, and connection of precast headwalls to precast culverts. seems to be very restrictive to ABC. is this intentional?	The design service life of epoxy dowels is uncertain. In ABC, there are many connections which do not rely on post-installed adhesive dowels.
2-44	**West Region**	**12.5.1 Post-installed Adhesive Dowels in Concrete** In the last paragraph it is not clear what this means. 25% of factored loads? how is factored resistance to be calculated for this check? A23.3? The last statement seems to completely remove the 25% limits, and allow full factored code strength. is this the intention? any issues mixing A23.3 resistance factors with S6 loads?	This section acknowledges that there is a wide range of practices for calculating the resistance of an adhesive dowel. A23.3 Annex D and the next version of S6 will contained provisions to check all failure modes of dowels. This section will be revised in the next version of the manual.

Comments received by Email			
Comment Number	**Organization**	**Comment**	**Response**
2-45	**West Region**	**14.3 Metal Culverts** Environmental factors shall be considered while checking the suitability of site for using SSPC. The thickness of SPCS and other steel components shall be determined according to CHBDC to ensure the culvert is structurally sound until the end of the design life. The design shall account for environmental conditions that exist at the site or are likely to exist during the design life of the structure, and the anticipated steel material loss during the design life. The MTO Gravity Pipe Design Guidelines and the CHBDC shall be referenced for guidance. Notwithstanding the requirements above, the thickness of the SPCS shall not be less than 5 mm.	Thank you, this has been corrected.
2-46	**West Region**	Poles" and related special provisions. **16.5 Pedestrian, Bicycle and MUP Bridges** 23? This section provides guidelines for the design of pedestrian and bicycle bridges. Pedestrian, Bicycle and MUP bridges shall be designed following CSA S7-23 – Pedestrian, cycling, and multiuse bridge design guideline. CSA S7-25 relies heavily on CSA S6 (CHBDC). In addition to those requirements, the requirements of Section 16.5.1 shall be followed.	Thank you, this has been corrected.
2-47	**West Region**	**18.2.1 Cantilever Static Sign Supports** RE: 48mm^2, maybe note that this is the butterfly max, maybe also give the number for a one-sided sign.	Thank you, the text has been updated.
2-48	**West Region**	•AADT is defined in section 1 – my understanding is this includes both bridges of a twin site. Section 2.5.3 (Seismic Importance) has a note that clarifies that AADT includes twin bridges. On a recent project the DBer tried to claim AADT for the purposes of section 12 was directional (ie EB was separate from WB) because section 12 didn't include a similar statement as section 2.5.3. Is it worth clarifying the AADT definition in section 1 to include twin sites?	Wording added to clarify AADT in both directions. S6-19, Section 12, clarifies which AADT to use for barrier design - 1-way traffic uses only that direction AADT, but it receives a Kh factor to end up in the same place.
2-49	**West Region**	SM 2024 Section, 8.7 General Notes #14-15: If the bridge is integral, … (no mentioning of exception of buried diaphragms & encased girder ends) If the bridge is semi-integral: all structural steel surfaces, except diaphragms, shall be coated as follows: from the ends of the girders to 600 mm beyond the front face of the abutment.	Since the girder projecting into the concrete is only coated for 100 mm from the surface, there should not be any diaphragms within the coated distance.

Comments received by Email			
Comment Number	Organization	Comment	Response
2-50	**West Region**	Section 8.1.3 Protection of Steel, 2nd & 3rd bullet point: For integral abutment bridges all structural steel surfaces, except diaphragms buried in concrete (what about encased girder ends ?) For semi-integral bridges all structural steel surfaces, except the areas of girders and diaphragms encased in concrete	In semi-integral abutments, the assumption and experience is that there is no direct leakage onto the diaphragms, therefore they do not need require coating. In contrast, the girders are subject to runoff and accumulation to debris over bearings.
2-51	**West Region**	Also the language on diaphragm in OPSS: OPSS 911.07.04.02.02 New Structural Steel All new structural steel including diaphragms, excluding surfaces in contact with concrete and the faying surfaces of bolted joints, ….., shall be coated…	The note on the drawing, specific to the particular situation on the bridge, takes precedence over the specification in the General Conditions.
3-1	**Entuitive**	2.1.2: This clause is restrictive for detailed structural design approaches and will tend to stifle innovation, particularly in mixed use bridges. It is the writer's view that 3D modelling of bridge structures will be the norm in the near future if it is not already, and that the more detailed approaches capture unusual behaviour better, reducing risk, and should be encouraged.	While we agree with this position that 3d models will become more widely used, there is little guidance on how to properly model bridges and validating the models is a challenge. MTO has adopted this position to protect for future rehabilitation or widening and change of functional use which have been routine for past slab-on-girder bridges.
3-2	**Entuitive**	Table 2.4.1 Concrete strength: Other jurisdictions, notably Alberta, routinely use 45 MPa transfer, 70 MPa 28 day strength. The higher transfer strength in particular is useful in maximizing the usefulness of the NU girder and is known to be available in Ontario precast facilities. It is suggested that the concrete strengths be adjusted.	These values have been determined based on extensive discussion with the precast industry, and is specific to the cement sources and mixes available in Ontario.
3-3	**Entuitive**	2.4.5 2) It is agreed that this is a good clause, but it appears to be in conflict with 8.15.1.5 in CHBDC 2019 which requires the reinforcing to be hooked over the longitudinal reinforcement.	The section has been updated.
3-4	**Entuitive**	3.3.1: Caisson shaft design – the restriction of spiral or hoop tie spacing to 150 is expensive and hard to justify based on the concrete column approach. The building sector uses the tied column provisions which would allow 300 or 400mm spacing.	The spacing requirements of spiral are based on CHBDC 8.14.4.2. When 25M or larger longitudinal bars are using in the compression members, the maximum spacing of 150mm is required. This requirement has been already confirmed for feasibility and constructability with industry.
3-5	**Entuitive**	3.3.1.3: The requirement for a permanent casing for a wet shaft does not seem consistent with practice and will impede soil/caisson bond and the use of caissons for combined support of excavation and permanent wall design. It is suggested instead that the use of drilling fluids to stabilize the shaft be emphasized.	The clause is updated to use a permanent or temporary casing for a wet pour.

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Comments received by Email			
Comment Number	Organization	Comment	Response
3-6	**Entuitive**	5.3.1: While the 40m limit is likely close to the practical limit, it is not clear why it is added since the forces from structural movement are to be calculated and addressed. We approach this type of structure as a rigid frame and completed the design on that basis.	The section has been revised and now refers to Section 11 Rigid Frames.
3-7	**Entuitive**	5.2.1.4: •For clarity, is 15M@150 horizontal and vertical now the minimum reinforcement? •Effective shrinkage compensating concrete mixes have been hard to obtain. Please indicate the acceptable shrinkage parameter to assist in specifying this item. •Please specify the maximum length between construction joints. Is it 12m? •Can well detailed contraction joints, similar to the barrier wall contraction joints, be used in lieu of construction joints to aid schedule. •Is there a minimum time between pours if the construction joint rather than a contraction joint is required.	Revised to clarify that the 150 mm spacing applies to horizontal reinforcement. Due to the thickness of the walls, we don't see contraction joints as a practical alternative. After further discussion, the reference to shrinkage admixtures has been removed. The time required to strip forms at the CJ will be sufficient - perhaps 3 days.
3-8	**Entuitive**	5.2.2: Jacking with live load should be assumed to be the typical case.	This section has been updated to incorporate this suggestion.
3-9	**Entuitive**	7.3.1 p) Avoidance of couplers is surprizing. The typical set up ensure no lift off of the first stage wedges. The couplers allow for a much more compact staging and substantially less complex detailing and congestion	The feedback from industry is that couplers are costly and not preferred for several reasons, including the need to place the strands into the ducts prior to placement. From a design and detailing perspective, they create a 'dead zone' with no prestress across the section, and therefore require substantial mild reinforcement to bring across the coupler. Designs we have seen with lapped tendons appear more compact and less congested.
3-10	**Entuitive**	7.3.5: Anchorage slip can be controlled to smaller numbers and this can be critical for shorter strands. It is suggested that this table be provided for guidance rather than an obligation.	Anchorage slip values provided are industry standards for bridge construction and often built into the seat of the jacks. After consulting with suppliers, we have decided to keep the long established values.
3-11	**Entuitive**	8.1.2 f): It is assumed that this clause is to ensure the exterior girder would not be deficient if the bridge were widened, which is understood but the reverse would not occur. It is suggested that all girders are to have the same profile and that the exterior girders cannot have less capacity than the interior girders.	We agree. The clause has been adjusted accordingly to make the intent more clear.
3-12	**Entuitive**	9.5.3: Please add a sketch to clarify the suggested added dowels	This will be added to the next version of the manual.
3-13	**Entuitive**	12.5: It is not clear why cementitious grout bonded anchors are not permitted. Cementitious grout is commonly used for anchorage of bearing masonry plates and cementitious grouts generally work better than epoxy when the gap in the hole is larger, allowing more tolerance in placement.	There was a period in time when MTO used cementitious grouted dowels. The quality of the grout can be more variable and MTO has adopted this policy for consistency, predictability and inspection across a large inventory of structures.

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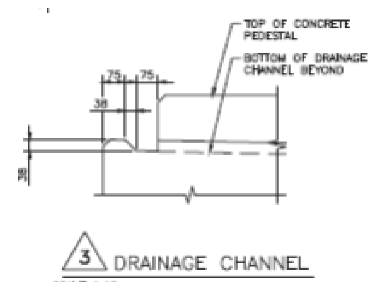
Comments received by Email			
Comment Number	**Organization**	**Comment**	**Response**
3-14	**Entuitive**	12.5.1: Please clarify that the 25 dia limit on dowels is limited to reinforcing steel and not anchors. Large diameter bars are typically used in jacking bracket design and some reviewers may confuse the anchors for dowels and object.	We removed the reference to anchors.
3-15	**Entuitive**	13.3.3.2: It is somewhat surprising that the use of plain bearings for permanent load applications is still permitted. In practice we do not see them as useful beyond the temporary application at locations where the girders will be cast into concrete after erection given their long history of excessive deformation.	The section has been revised to preclude them from use in permanent applications.
4-1	**MTO**	MTO Policy Memo 2020-04, relating to seismic evaluation for bridge rehabilitation, appears to use earthquake frequency more than specified in Section 4.11.	CHBDC Exception added to allow increased earthquake frequencies for shorter remaining service life.
4-2	**MTO**	6.2.3 Rust control- consider providing narrower rust scuppers to avoid excessive sizing requirements for pier caps and shafts. The detail in Fig 6.2.2 takes up 500mm of width. For P/S girder with two bearings this can result in a particularly wide cap. Westchester Bourne alternate detail below. Or consider providing language that permit modifications to the drainage channel to reduce width.  	Added a sentence to require the rust dams be at least 75 mm wide.
4-3	**MTO**	7.3.1 l) I can appreciate that bond style anchorages are lower durability compared to fully encased grouted systems, but I think there will still be cases where it practically makes sense to use them (E.g tight C/C spacing). If they're embedded deep in a section that is waterproofed or in a benign environment, is it really a concern or is it good enough? Suggest adding "unless approved by the Head of Structural Section"	The section has been revised accordingly.
4-4	**MTO**	7.3.5 Consider adding the following: "The designer shall specify power seating on short tendons when anchorage slips losses are a major contributor to the total prestress loss of the tendon" May also want to add restriction on two-end stressing per FHWA "Stressing from two ends shall not be specified when the calculated elongation is less than the length of the wedge grip". Also "Curved bar tendons are not permitted"	The section has been revised.

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Comments received by Email			
Comment Number	**Organization**	**Comment**	**Response**
4-5	**MTO**	7.3.9 A) 12) Min plastic duct wall thickness 1mm. Consider increasing this to 2mm and referenced PTI/ ASBI M50.3-19 Table 4.1. E/F) Legacy clause on stressing sequence.. {Placing concrete in recesses for longitudinal tendon prior to grouting long. tendon} -> reverse order of operations. Especially pourbacks on many transverse recesses can take a while. Want grouting before pouring recesses.	Min thickness has been deleted because it is covered in OPSS 910. The sequence has been revised.
4-6		8.1.1 suggest rewording second para. Sounds like high levels of cl- are ok as long as UWS can dry	Reworded accordingly to make the intent the more clear.
4-7	**MTO**	8.1.2 k) Huck bolt fasteners. This seems like a strange legacy clause that is out of touch with current practice and does not work well with DB. Suggest deleting entirely. n) Reconcile 1.5mm loss and 2mm in Fig 8.1.1.	k) has been deleted and n) has been corrected.
4-8	**MTO**	Fig 8.3.2. Galvanized pipe at drainage location not great from a galvanic corrosion point of view. It also doesn't show how this is attached. Many designers are tapping a threaded pipe in. I'm not a fan of that detail as it just doesn't seem like a good idea from a fatigue point of view (creating a notch). I've also see the counterbored and flanged pipe that is caulked in. Would probably still want to radius the counterbore a bit. Vent/ drain is best made of plastic/ inert and "glued in" so you don't get it popping out and avoids the galvanic cell.	We agree. Drain pipe through steel tub girders is revised to plastic per MTO practice.
4-9	**MTO**	Fig 10.6.2 I know it came from the memo, but I don't like the railing stopping short of the end of the platform or edge or retaining wall. The slopes can often be densely vegetated and it gives a false sense of the end of the grade difference. Suggest showing the guards to the end.	This will be considered in future revisions to this standard.
4-10	**MTO**	Fig 12.2.2 show as 50mm gap to match earlier revisions?	The figure is pictorial to convey the extent of premium reinforcement, and would apply regardless of whether the median gap is 50 mm or 3 m.
4-11	**MTO**	13.3.2.7 "Applied horizontal loads should be consistent with applied axial loads" This is not really how this table works. Max horizontal loads generally don't coincide with the transitory load cases reported in the table which are selected to show the max and min axial loads, most often ULS 1 and 2.	The section has been updated to suggest that additional rows may be added to the table.