ASCE-INDOT
STRUCTURAL SUBCOMMITTEE
MEETING NO. 46 MINUTES
January 12, 2010

The meeting was called to order at 9:05 am by Anne Rearick. Those in attendance were:

Anne Rearick	INDOT, Structural Services
Tony Uremovich	INDOT, Structural Services
Ron McCaslin	INDOT, Structural Services
Naveed Burki	INDOT, Structural Services
Brian Harvey	INDOT, Program Development
Jim Reilman	INDOT, Construction Management
Mike McCool	Beam Longest & Neff, LLC.
Mike Wenning	American Structurepoint, Inc.
Mike Halterman	USI Consultants, Inc.
Burleigh Law	HNTB Corp.
Jason Yeager	Gohman Asphalt Company
Don Bosse	Prestress Services, Inc.
Bob McCullough	Purdue University
Michael Eichenauer	Butler, Fairman and Seufert, Inc.

In addition to the attendees, these minutes will be sent to the following:

George Snyder	INDOT, Structural Services
Ron Heustis	INDOT, Construction Management
Greg Klevitsky	INDOT, Structural Services
Bill Dittrich	INDOT, Program Development
Tony Zander	INDOT, Materials and Tests Division
Keith Hoernschmeyer	Federal Highway Administration
Dick O'Connor	RQAW Corporation
Troy Jessup	R. W. Armstrong
Steve Weintraut	Butler, Fairman and Seufert, Inc.

A meeting agenda had previously been distributed and the following items were discussed:

1. The October 22, 2009, meeting minutes were approved as written, and have been placed on the INDOT website.

2. The Bridge Design Conference is tentatively set for July 27 in the Indiana Government Center South Conference Rooms A, B, and C. The time will be from 8:30 to 4:30.

3. The pavement ledge details were discussed in relation to the problems that occurred on SR 101. It was suggested to eliminate the #5 bent bar that runs from the end bent, through the pavement ledge and into the approach slab. Jason felt that this was not being installed in the field correctly because of constructability issues. It was suggested that threaded tie bars be used as has been done in the past (see Attachment No. 1). Changes to this detail include using #4 threaded tie bars in lieu of #6 and space them to match the top longitudinal steel in the deck. The spacing of the threaded tie bars would be 24" so if the top longitudinal steel was 8", the threaded tie bars would lap with every third deck bar. The threaded tie bar lengths would alternate

between 3'-0" and 5'-0" lengths across the approach slab as detailed. Jim Reilman will find a bridge to try this method on.

4. Tony Zander was not present to provide an update on the semi-lightweight concrete specification.

5. Tony Zander was not present to provide an update on the self consolidating concrete specification.

6. Bob McCullough presented a slideshow on construction loadings that he was going to use at the County Bridge Conference. He presented the general requirements as covered in the INDOT specifications and in LRFD. He also showed slides of failures in other states and their code requirements. His conclusions are that INDOT needs to address temporary lateral bracing requirements, evaluate their approach to temporary structure requirements including loads, specifications, contractor requirements and costs, and to try to limit their exposure. Mike McCool passed out a handout (see Attachment No. 2) describing construction loading requirements that his firm currently uses.

7. Burleigh presented his finding for the overhang criteria for all beam types. His findings determined that the 0.45 X the beam spacing and 5' max overhang are good but the 0.85 X the beam height should be eliminated. It does not control safety. Mike Halterman will present this to his Group 8 Loading Committee.

8. Mike McCool again discussed limiting the deck steel spacing to 8" maximum in both the longitudinal and transverse direction. Anne suggested he discuss with Randy Strain to get his comments.

9. Anne has passed the 400k collision load to the Group 7 Substructure Committee for their recommendations.
   Steve emailed his recommendations to Anne as follows:

   a. Abutments: Due to the existence of soil behind abutment walls (including MSE), consider abutments exempt from meeting the substructure protection requirements for highway overpasses and for RR overpasses with abutments more than 25 feet from the centerline of railroad tracks.
   b. Foundation Requirements: Exempt spread footing and pile/drilled shaft foundations from the 400 kip load analysis since extra resistance is provided by passive soil pressure, friction and pile structural capacity.
   c. Low volume roads: Bridges spanning over roadways with low design speeds and ones that have a design speed greater than 40 mph, but ADTT less than or equal to 250, should be exempt from meeting the 400 kip load analysis or providing TL-5 pier protection.
   d. With some slight modification, I would recommend TL-5 pier protection details similar to those developed by the FDOT.

10. Jason discussed his experience with hydrodemolition. He would like to include payment for additional overlay on hydrodemolition work. Anne is still discussing with INDOT.

11. Tony passed out a handout on the beam fillet details (see Attachment No. 3). The group liked the details. Do not include the plan details (sheets 2 of 2 for both beam types), only the section details. On Figure 61-4C, the following changes need to be made:

Eliminate the concrete insert detail
Flip the angle on the left flange (leg up) to match the angle on the right flange

The next meeting for the INDOT Structural Subcommittee is scheduled for 9:00 am on April 13, 2010, in a room to be determined.

This meeting was adjourned at 11:20 a.m.

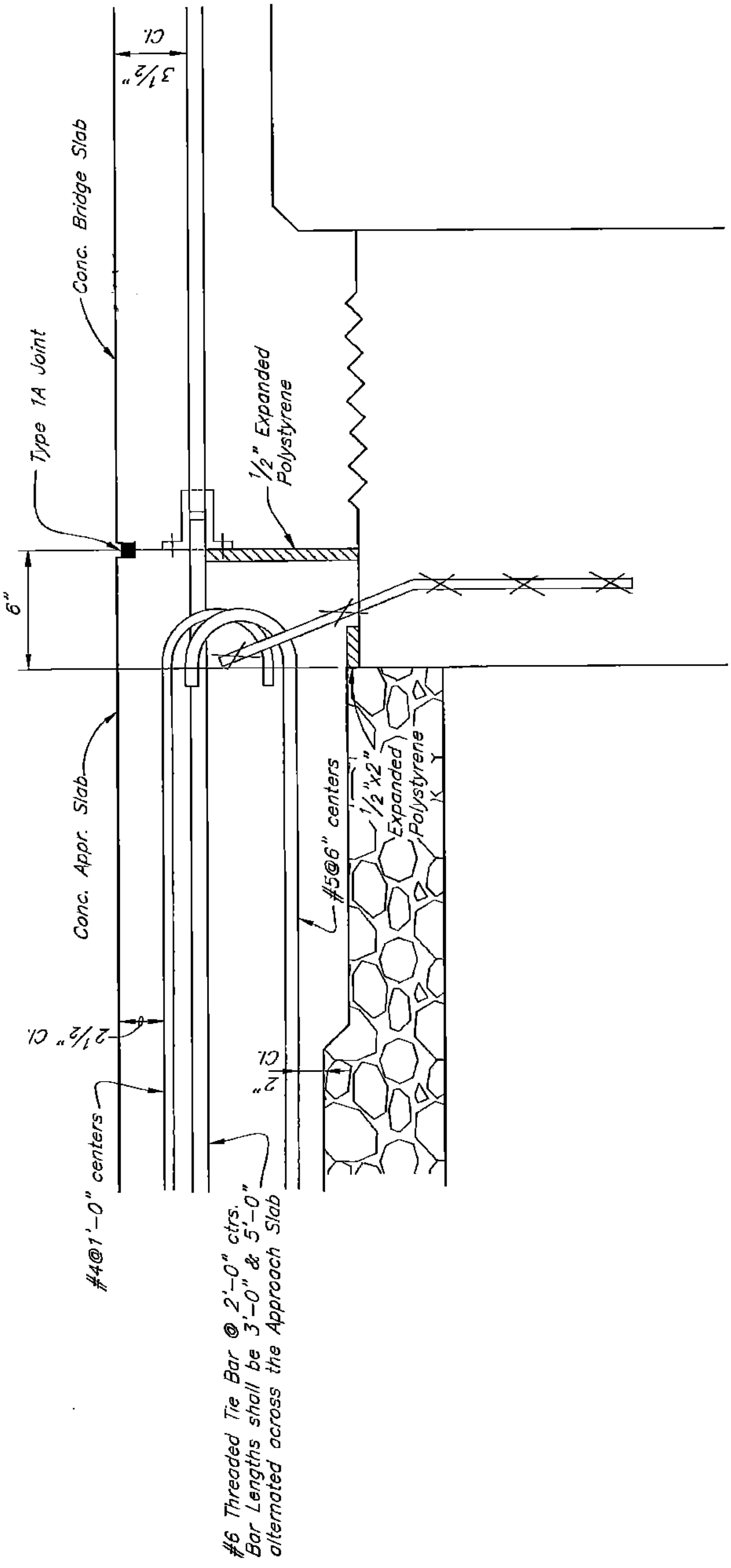
Respectfully submitted,
BUTLER, FAIRMAN and SEUFERT, INC.

Michael Eichenauer, P.E.
meichenauer@bfsengr.com

ME:me

Attachments

ATTACHMENT NO. 1



PAVEMENT LEDGE DETAIL

Not to Scale

Construction Loads

During construction, the steel superstructure is the most vulnerable to horizontal loading. During this stage, the reinforced concrete deck is not present to distribute the horizontal loads to all of the beams. All of the horizontal distribution and bracing is a result of the diaphragms. This lack of horizontal support, coupled with additional horizontal loading from construction equipment and formwork can cause extreme loadings that are not checked with the typical in-service design calculations.

APPLIED LOADS

The capacity of the non-composite steel members shall be checked using the load factors for the following loads.

Component Loads (DC):
- DC1 – Concrete = 150 lbs/ft^3
- DC2 – Stay-in-place Formwork = 15 psf

Construction Dead Loads (CDL):
- CDL1 – Removable Coping Deck Forms = 15 psf
- CDL2 – Temporary Walkway = 15 psf – applied over a 2'-0" wide platform on outside of coping

Construction Live Loads (CLL):
- CLL1 – Construction Live Load = 25 psf extended the entire bridge width plus two feet outside of bridge coping over 30 feet longitudinal length centered on Screed Machine Load
- CLL2 –Screed Machine = 4500 lbs over 10 feet longitudinal length applied 6" outside of bridge coping.
- CLL3 – Vertical Railing and Walkway Load = 75 plf applied 6" outside of bridge coping over 30 feet longitudinal length centered on Screed Machine Load

Wind Load (WS):
- Calculated per AASHTO 3.8.1.1 (use 70 mph per AASHTO Temporary works manual Fig 2.1.

See Figure 1 and Figure 2 for the application of these loads. The angled bracket should be assumed to extend from the flange/web intersection to 6" outside of the edge of coping to maximize the horizontal force in accordance with AASHTO C6.10.3.4.

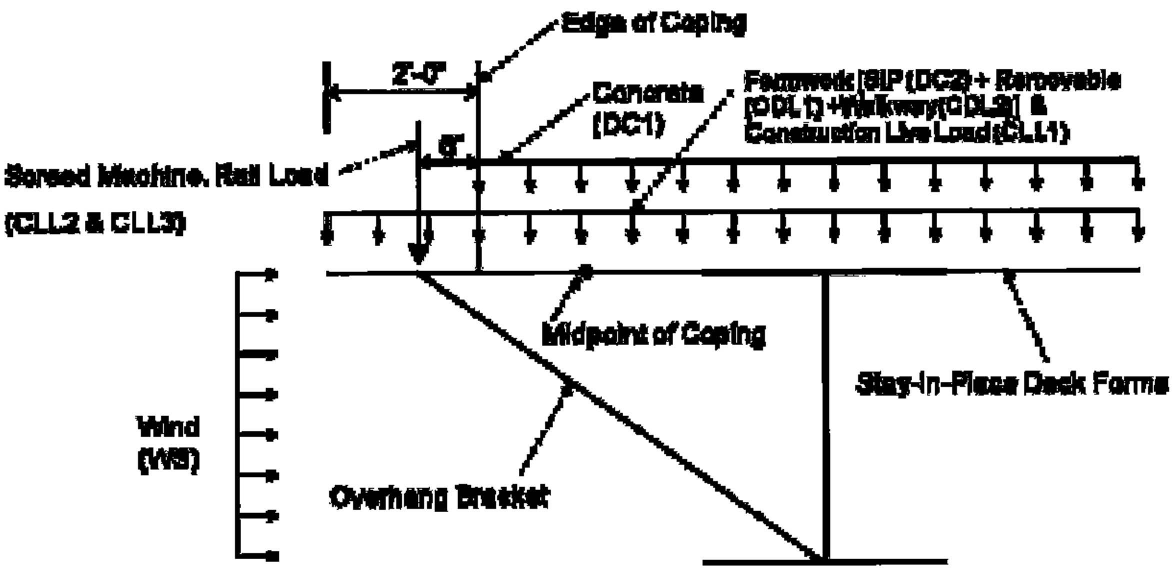


FIGURE 1

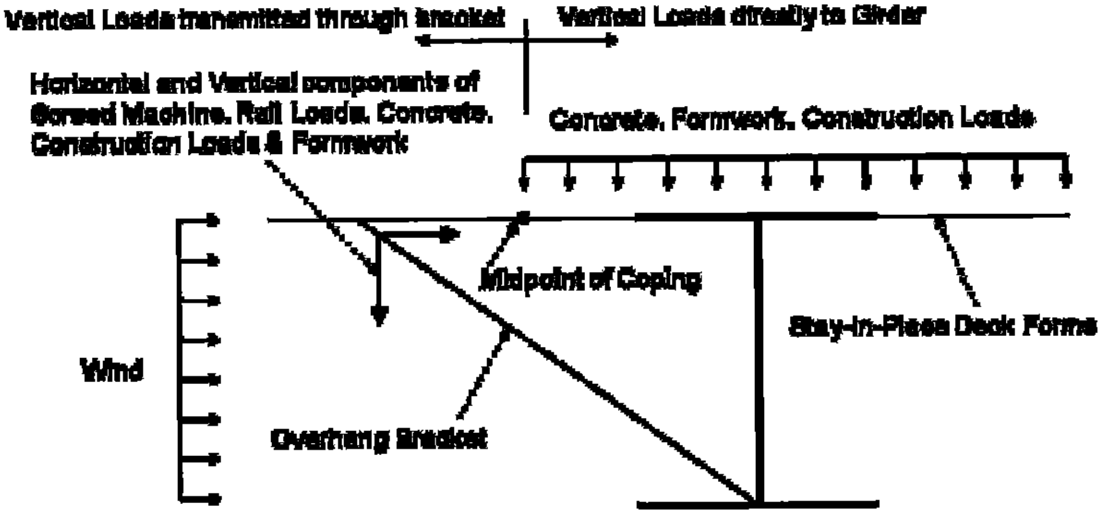


FIGURE 2

LOAD FACTORS

In accordance with AASHTO 3.4.2.1 and AASHTO 3.4.2.2, the load factors for loads during construction are:

Strength I –	1.25 * Component Loads + 1.5 * Construction Dead Loads + 1.5 * Construction Live Loads
Strength III –	1.25 * Component Loads + 1.5 * Construction Dead Loads + 1.25 * Wind Loads
Strength IV –	1.5 * Component Loads + 1.5 * Construction Dead Loads
Strength V –	1.25 * Component Loads + 1.5 * Construction Dead Loads + 1.35 * Construction Live Loads + 0.40 * Wind Loads
Service I –	1.00 * Component Loads + 1.00 * Construction Dead Loads + 1.00 * Construction Live Loads + 0.30 * Wind Loads

To check slip critical connections in accordance with AASHTO 3.4.1, the load factors are:

Service II –	1.00 *Component Loads + 1.30 * Construction Dead Loads + 1.30 * Construction Live Loads

LIMIT STATES

After the composite deck is cast on the steel superstructure, the top of the beam is considered to be braced by the deck. The deck works as a large diaphragm, continuously distributing horizontal loads to each beam. Because the beam flanges are considered continuously braced, flange lateral buckling need not be considered. However, during construction there is no rigid diaphragm to continuously distribute the horizontal loads. The deck forms are ignored for this function and only the diaphragms are recognized as acceptable to distribute the horizontal loads. In accordance with AASHTO 6.10.3.2.1, AASHTO 6.10.3.2.2 and 6.10.3 seven limit states should be examined:

Limit State 1 – Yielding Limit State Check – This check ensures that the maximum combined stress in the compression flange will not exceed the specified minimum yield strength of the flange times the hybrid factor.

$$F_{bu}+f_l \leq \varphi_f R_h F_{yc} \qquad \text{(AASHTO 6.10.3.2.1-1)}$$

Limit State 2 – Lateral Torsional Buckling and Flange Local Buckling Check – This check ensures the member has sufficient strength with respect to lateral torsional and flange local buckling based limit states, including the consideration of flange lateral bending where these effects are judged to be significant.

$$F_{bu}+1/3f_l \leq \varphi_f F_{nc} \qquad \text{(AASHTO 6.10.3.2.1-2)}$$

Limit State 3 – Flange Lateral Bending Check – This check ensures that the geometry of the section and overhang do not cause excessive horizontal stresses in the flanges.

$$f_l \leq 0.6_{yf} \qquad \text{(AASHTO 6.10.1.6-1)}$$

Limit State 4 – Web Bend Buckling Check – This check ensures that the theoretical web bend-buckling will not occur in construction.

$$F_{bu} \leq \varphi_f F_{crw} \qquad \text{(AASHTO 6.10.3.2.1-3)}$$

This check need not be performed if the web is compact or non-compact per AASHTO 6.10.6.2.3.

Limit State 5 – Discretely Braced Flange in Tension Check – This check ensures that the stress in the flange will not exceed the specified minimum strength of the flange times the hybrid factor during construction under the combination of the major-axis bending and lateral bending stresses due to factored loads.

$$F_{bu}+f_l \leq \varphi_f R_h F_{yt} \qquad \text{(AASHTO 6.10.3.2.2-1)}$$

Limit State 6 – Lateral Girder Rotation Check (Service I) – This check ensures while the deck is being sequentially poured, the exterior beam does not rotate, resulting in excessive overhang deflections. This can adversely affect finished grades when the screed rail is placed at the end of the overhang. The eccentric loads should be applied to the exterior beam to determine the amount of torsion each load causes in the beam. Using the torsion, the horizontal forces in the top flange (causing deflection outward) and bottom flange (causing deflection inward) should be calculated. Each flange should be analyzed as a continuous beam over supports, where the supports would be the diaphragm spacing. The loads should be applied to match how the deck will be poured, most likely with the concrete and screed machine starting on one end, with the other end virtually unloaded. After the analysis, the inward deflection of the bottom flange and outward deflection of the top flange should be used to determine the rotation of the beam. This rotation is directly related to the rotation of the coping as shown in Figure 3. The maximum rotation of the coping should be limited to 0.20 inches.

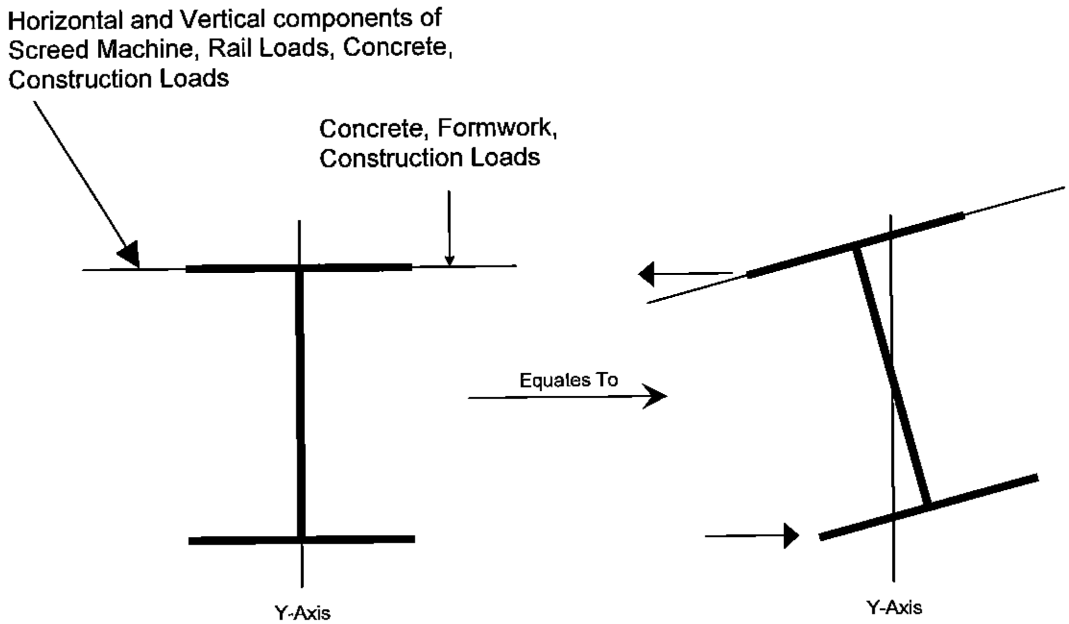
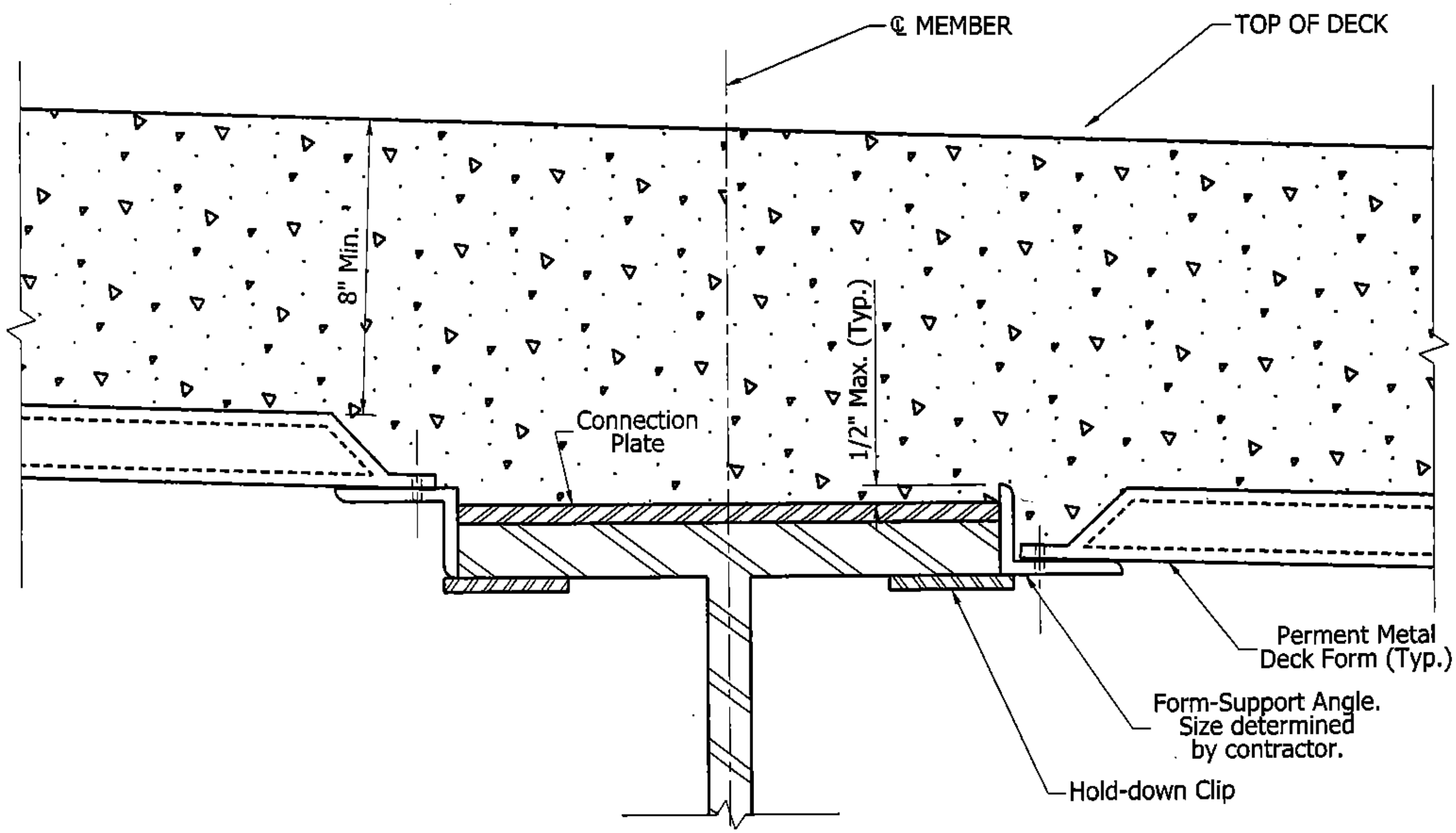


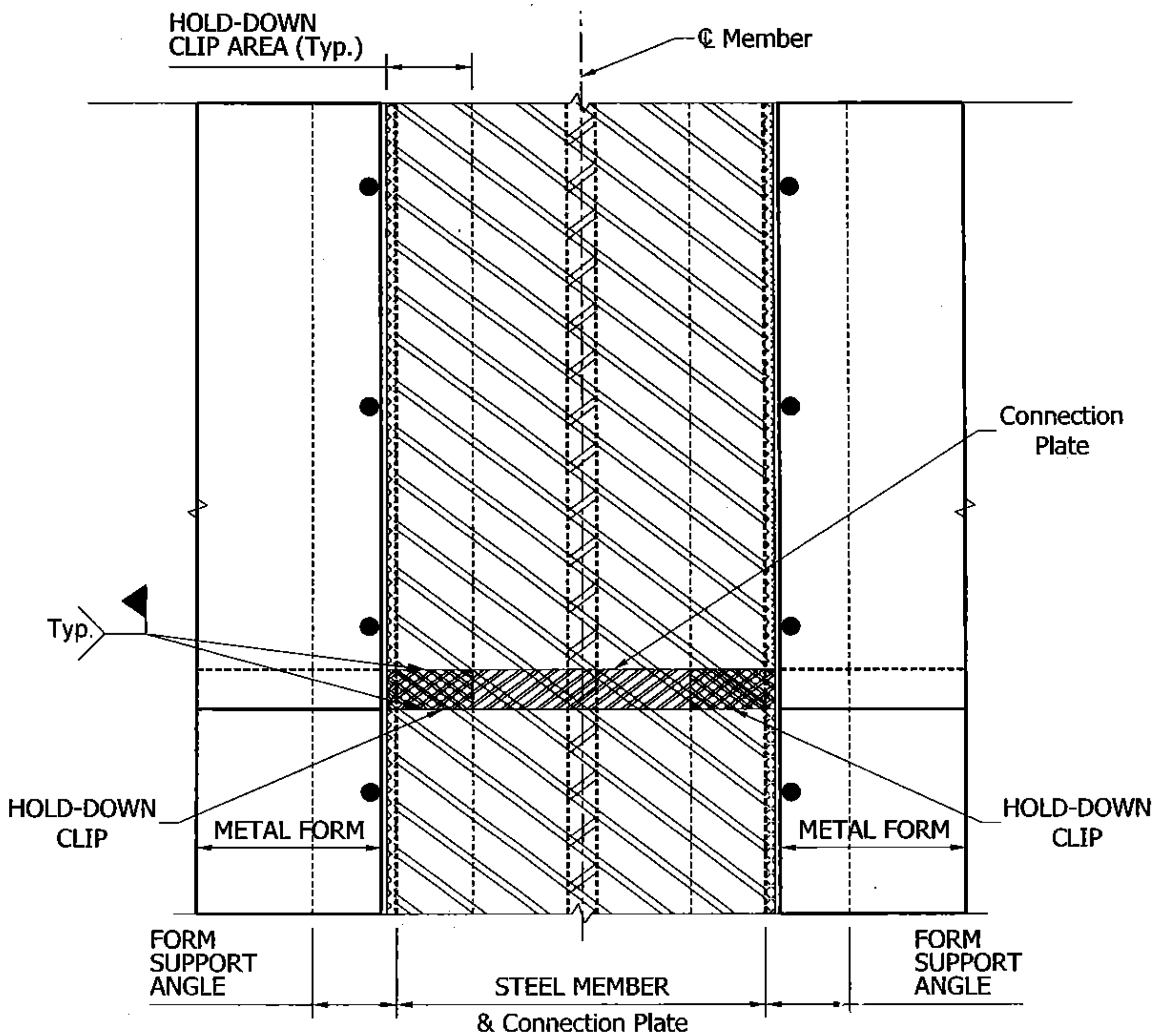
FIGURE 3

Limit State 7 – Diaphragm Slip Critical Bolt Check (Service II) – This check ensures that the connection used to attach the diaphragms to the webs of the steel members is adequate to resist the moment caused by the lateral rotation of the girder and horizontal force caused by the overhang bracket.



FILLET TREATMENT FOR
STRUCTURAL-STEEL MEMBER

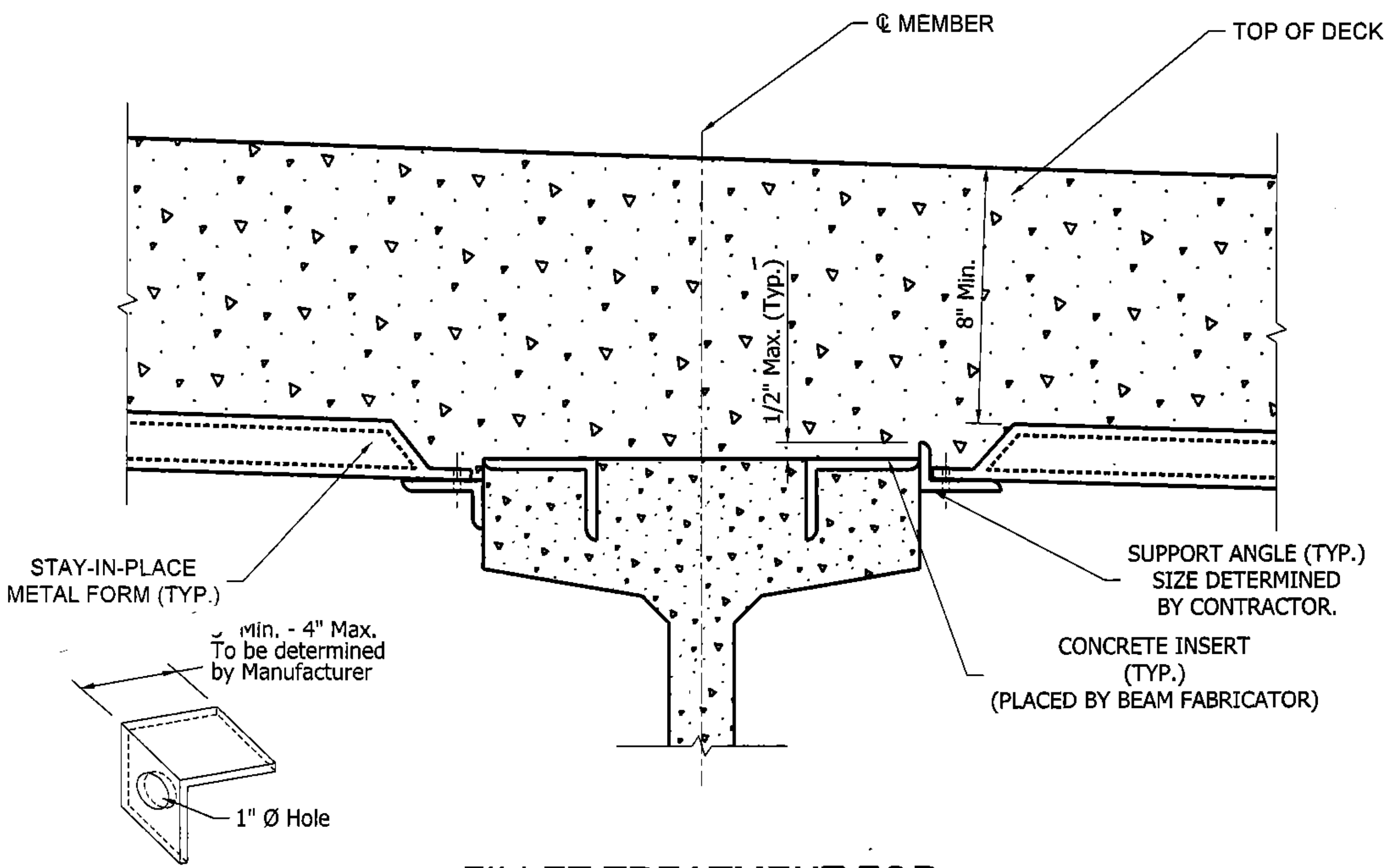
Figure 61-4B
(Page 1 of 2)



SELF-TAPPING SCREWS LOCATIONS
AT STRUCTURAL-MEMBER FILLETS

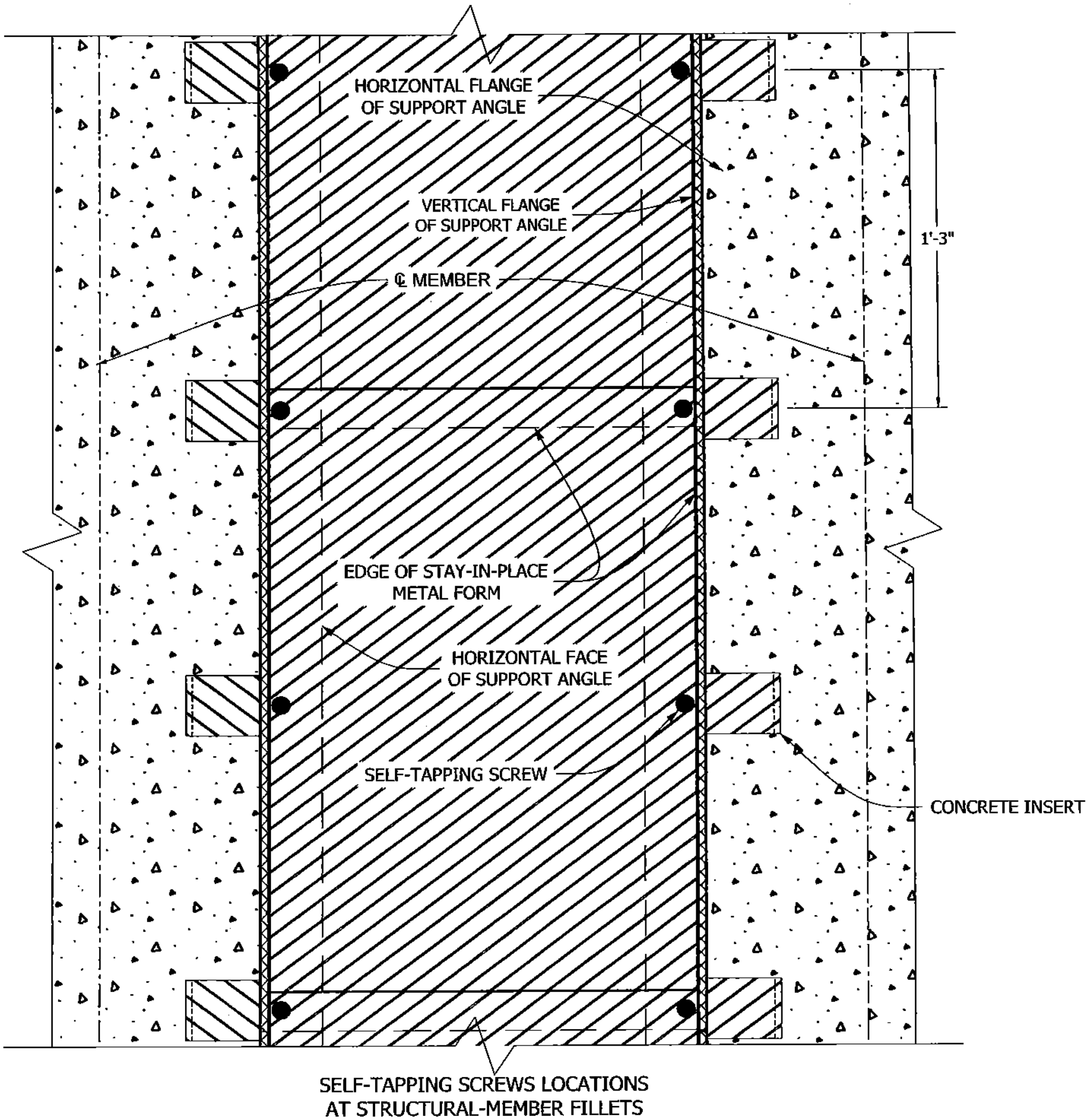
FILLET TREATMENT FOR STRUCTURAL-STEEL MEMBER

Figure 61-4B
(Page 2 of 2)



FILLET TREATMENT FOR
PRESTRESSED-CONCRETE MEMBER

Figure 61-4C
(Page 1 of 2)



FILLET TREATMENT FOR
PRESTRESSED-CONCRETE MEMBER

Figure 61-4C
(Page 2 of 2)