ASCE-INDOT
STRUCTURAL SUBCOMMITTEE
MEETING NO. 54 MINUTES
February 23, 2012

The meeting was called to order at 9:00 a.m. by Mike Eichenauer. Those in attendance were:

Randy Strain	INDOT, Structural Services
Anne Rearick	INDOT, Structural Services
Tony Uremovich	INDOT, Structural Services
Naveed Burki	INDOT, Structural Services
Keith Hoernschmeyer	Federal Highway Administration
Mike Wenning	American Structurepoint, Inc.
Burleigh Law	HNTB Corp.
Mike McCool	Beam Longest & Neff, LLC.
Troy Jessop	R. W. Armstrong
Mike Halterman	USI Consultants, Inc.
Celeste Spaans	Prestress Services, Inc.
Michael Eichenauer	Butler, Fairman and Seufert, Inc.

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

Ron McCaslin	INDOT, Structural Services
Jim Reilman	INDOT, Construction Management
Tony Zander	INDOT, Materials and Tests Division
Merril Dougherty	INDOT, Program Development
Brian Harvey	INDOT, Program Development
Jason Yeager	Gohmann Asphalt Company

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

1. The November 21, 2011, meeting minutes were approved as written, and have been placed on the INDOT website.

2. Randy Strain will write a specification for light weight concrete and present a draft to the committee at the next meeting. The issue for the precasters is mainly achieving the weight range in the specification. The INDOT Specification lists a weight range of 124.5 pcf – 130.4 pcf. The lightweight concrete wet to dry weight loss varies significantly by aggregate type. Designers who specify a weight of 130 pcf, will have a final weight range of 117 pcf – 123 pcf. Celeste will pass along information from other states to Randy. Designers need to have an understanding of the dry weight, final weight, and steel weight when specifying concrete.

3. The Bridge Design Conference subcommittee has not met yet. Mike McCool will set up a conference call soon to begin discussions for this year's conference. Suggested topics for next year include construction loading examples for steel and concrete beams, seismic design, and residual camber calculations. These and other topics will be discussed by the committee in the next coming months. The tentative date for the conference next year will be the week of July 23. Anne will check on room availability.

4. Mike McCool has developed a research plan to give to Professor Frosch on stirrup reinforcement in prestressed beams. This topic will be moved to the parking lot.

5. Naveed Burki questioned the use of articulated mats based on cost. It was pointed out that IDNR requires it on some projects for animal crossings. He suggested using riprap unless required to use articulated mats. And at the most, use articulated mats for what is required and switch to riprap for the rest of the spillslope. INDOT will work on a specification for the articulating mats.

6. Mike Wenning stated that there needs to be clarification in the MSE wall specification. The specification lists H when describing the wall height under all circumstances. However, the old special provision used to list H and H' to describe different design heights. Mike believes that the specification needs to list both H and H'. This will be discussed in the Standards Committee in March and will be revised officially in September.

7. Mike McCool passed out a handout (Attachment 1) suggesting changes for end bents by eliminating the #6 bar going into the floor. INDOT will look at the detail and possibly make modifications.

8. The socket detail for drilled shafts may need revisions. The current detail in the Design Manual shows a different diameter socket than shaft. This detail may remain and an additional detail showing the shaft and socket diameter the same may be included.

9. Mike Wenning passed out a handout (Attachment 2) on a spread footing retaining wall. His concerns with the LRFD design is the footing widths and buoyancy. Mike would like the group to review the spreadsheet and provide comments at the next meeting.

10. Mike McCool passed out a handout (Attachment 3) with new language for the wind loading. The Design Manual will be updated with this new language.

11. The group discussed the draft of the ASCE-INDOT Structure Committee Operations Requirements and suggested revisions and modifications. Anne Rearick will revise the document accordingly.

12. Mike McCool passed out a handout (Attachment 4) on end bent method B. Mike would like to use rebar for the anchor plate anchorage instead of a headed stud or anchor bolt.

13. Mike brought up issues with camber tolerances that will be discussed at the next meeting.

14. Burleigh Law suggested a needed criteria for lighting for box girders. He will send information to Anne.

The next meeting for the INDOT Structural Subcommittee is scheduled for 9:00 a.m. on May 22, 2012, in a room to be determined.

This meeting was adjourned at 11:15 a.m.

Respectfully submitted,
BUTLER, FAIRMAN and SEUFERT, INC.

Michael H. Eichenauer

Michael Eichenauer, P.E.
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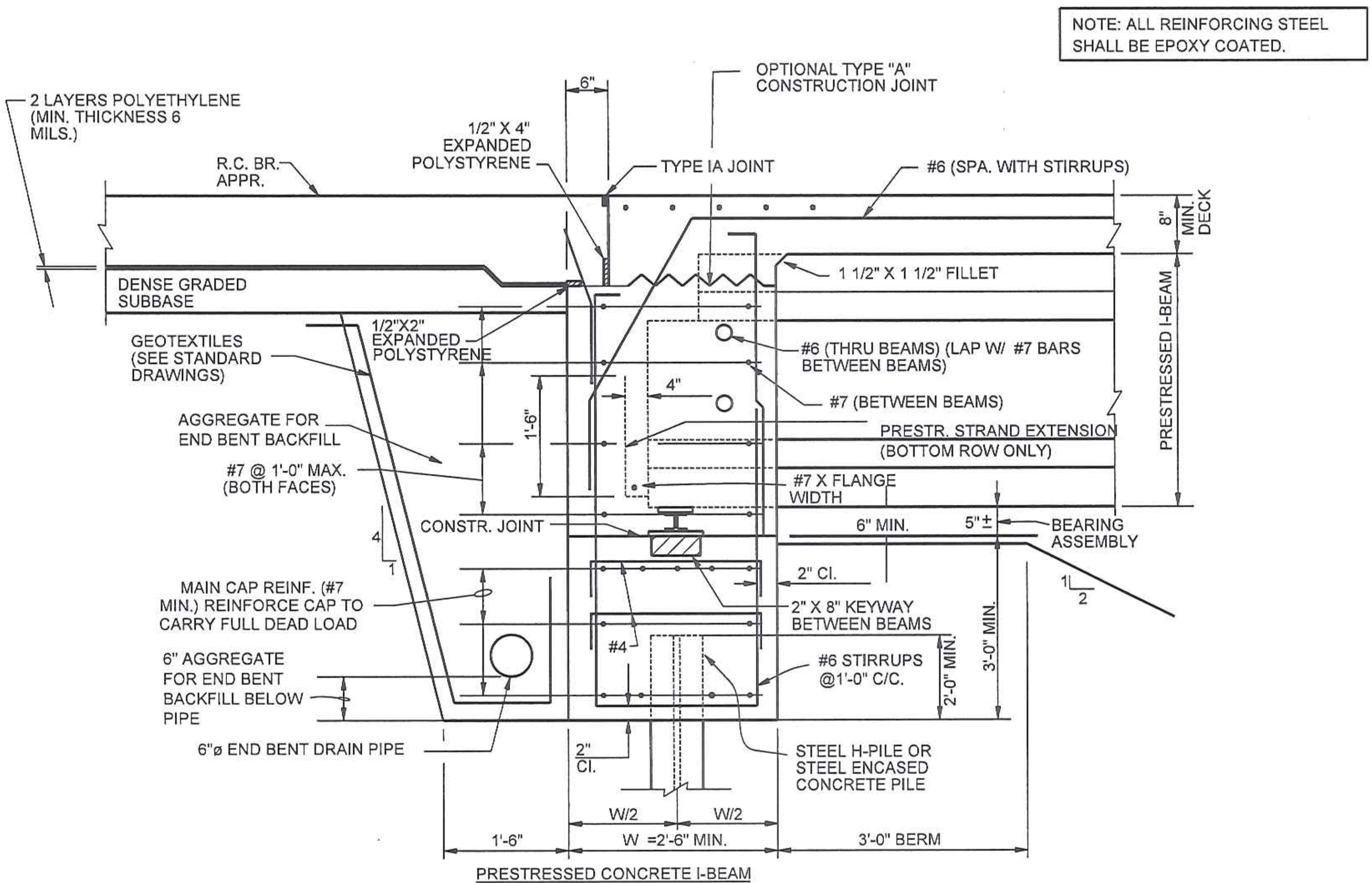
ME:me

Attachments

ATTACHMENT NO. 1



SUGGESTED INTEGRAL END BENT DETAILS
(Beams Attached to Concrete Cap, Method B)

Figure 409-2C
(Page 1 of 4)

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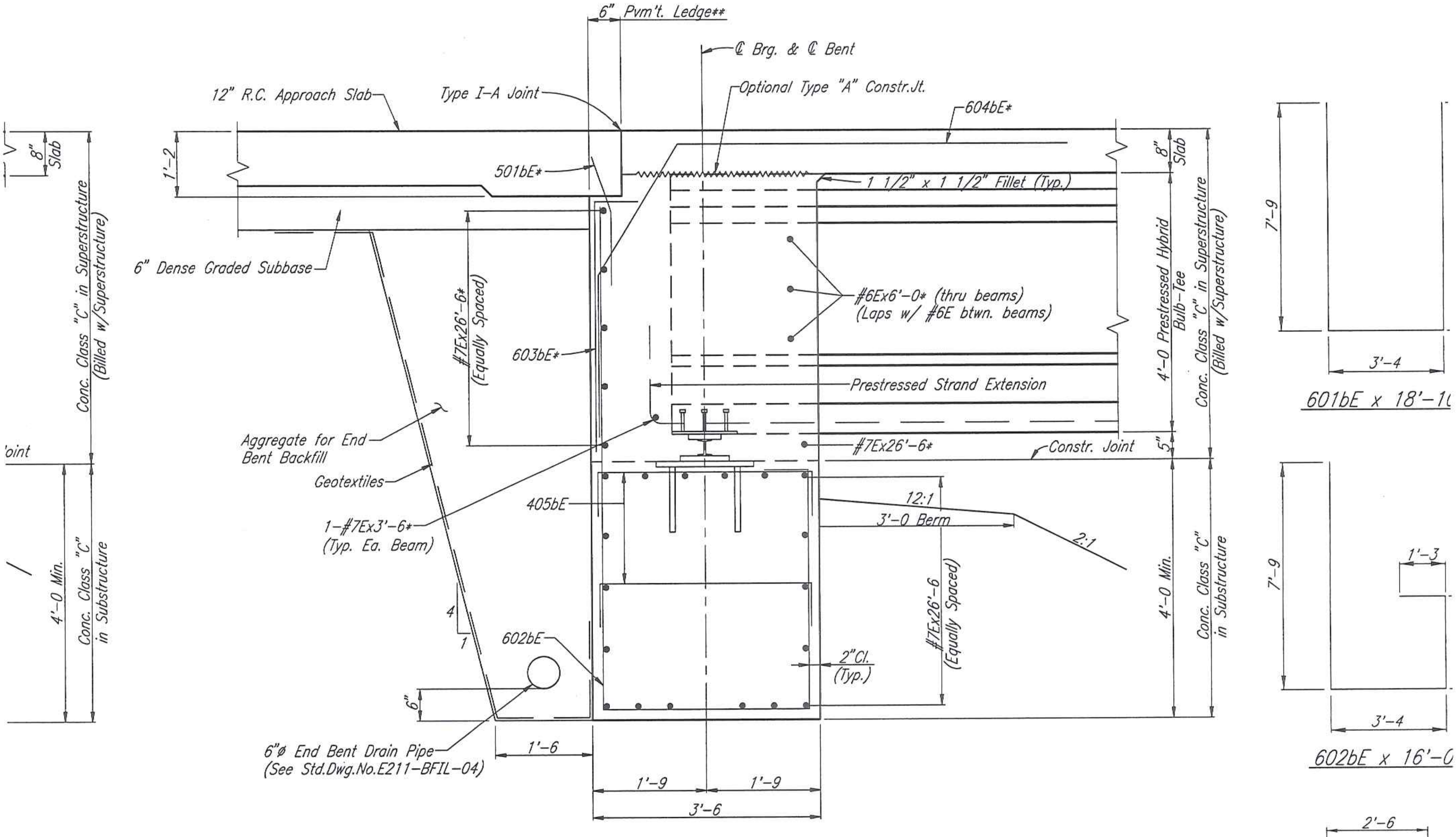
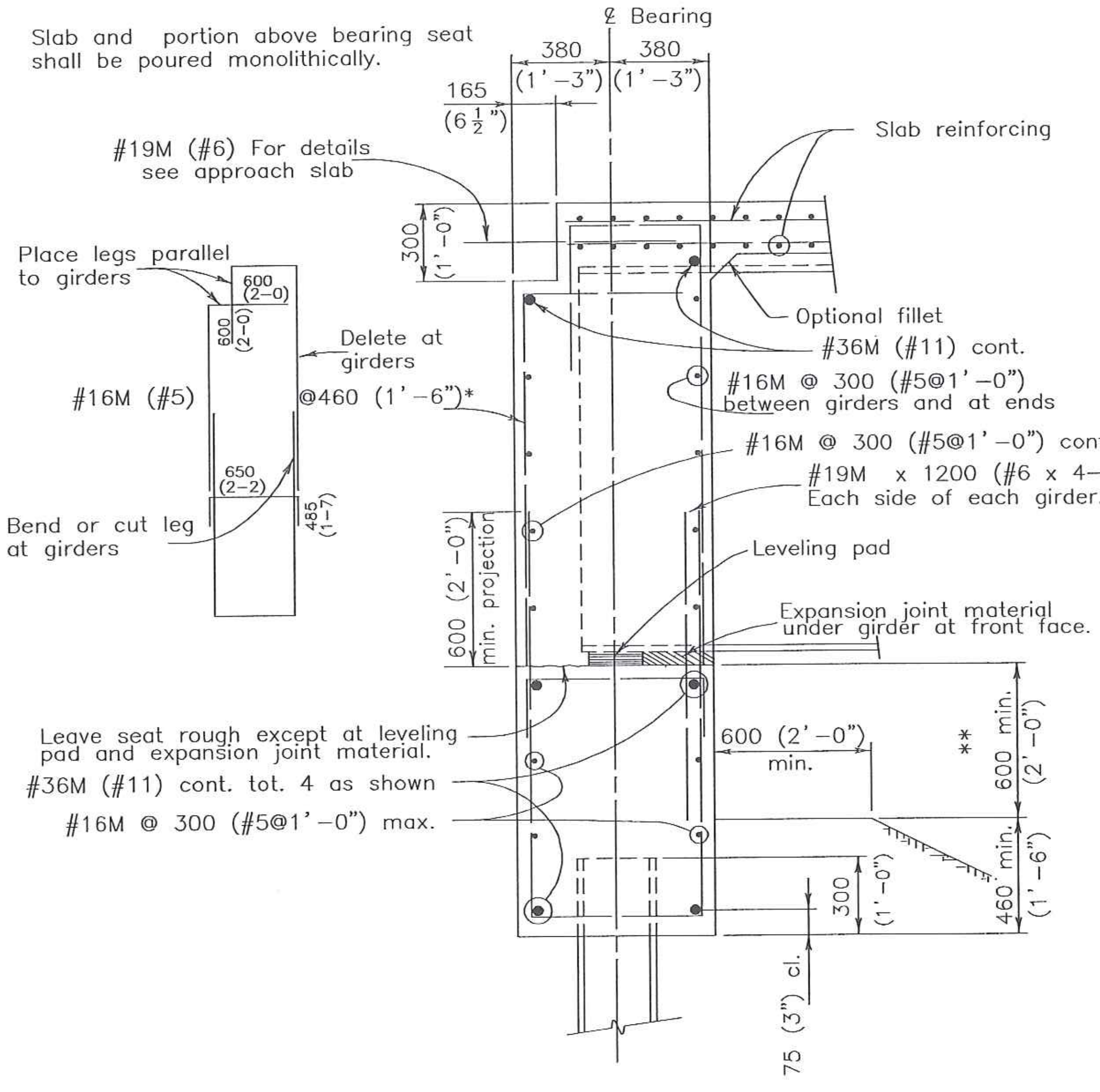


SECTION B-B

Scale: 3/4"=1'-0



TYPICAL ABUTMENT SECTION

Note: All abutment and wingwall concrete shall be Class D (Bridge)

Extend strands from the bottom of precast sections into abutment, anchor the bottom of steel sections to abutment with studs, bearing stiffeners, anchor bolts, or diaphragm gussets.

* 300 (1'-0") if structure length longer than 90M (300') or ** greater than 1050 (3'-6").

SOIL PROPERTIES		
Enter Bearing Material Type (ROCK or SOIL):	SOIL	
Enter FACTORED Bearing Resistance:	4,200	psf
Enter Angle of Surface Slope, β:	5.5	°
Enter Backfill Weight, γ_s:	125	pcf
Enter Backfill Internal Friction Angle, ϕ:	32.0	°
Enter Interface Friction Angle, δ:	17.0	°
Calculated Coefficient of Sliding Resistance, f:	0.50	
Enter Height of LL Surcharge, h_{eq}:	2.00	ft
Enter Depth of Soil Above Footing Toe:	6.80	ft
Enter Height of Passive Soil (from Ftg Btm):	0.00	ft
Enter Height of Water Table (from Ftg Btm):	4.00	ft

CONCRETE PROPERTIES		
footing f'_c =	3.0	ksi
F_y =	60.0	ksi
stem f'_c =	3.5	ksi
γ_c =	150	pcf

SOIL PRESSURE DATA		
θ =	90.0	°
k_a =	0.30	
k_p =	6.30	
P_a =	5,564	lbs
P_p =	0	lbs

WALL GEOMETRY		
Enter Wall Height (from Ftg Btm to Stem Top):	14.60	ft
Enter Stem Width at Top:	1.25	ft
Enter Stem Batter (i.e., 1:12 = 12, None = 0):	0.00	
Enter Total Footing Length:	11.75	ft
Calculated Footing Heel Length:	10.00	ft
Enter Footing Toe Length:	0.50	ft
Enter Footing Thickness:	3.00	ft
Calculated Stem Width at Base:	1.25	ft

SUMMARY		
H =	14.60	ft
H_{des} =	15.57	ft
L =	11.75	ft
Toe =	0.50	ft
Heel =	10.00	ft

SHEAR KEY		
Depth =	0.00	ft
Width =	2.00	ft (min)

OVERTURNING MOMENTS, M_o (STR I LOAD COMBO)

	P_a	x	Arm	x	$\gamma_{over.}$ / $\gamma_{bear.}$	=	Over. Moment	=	Bear. Moment
Earth:	4,347	x	5.19	x	1.50 / 1.50	=	34 k-ft	=	34 k-ft
Buoyancy:	-143	x	1.33	x	1.50 / 0.00	=	0 k-ft	=	0 k-ft
Surcharge:	1,168	x	7.79	x	1.75 / 1.75	=	16 k-ft	=	16 k-ft
$P_{a\_sliding}$ = 8.3 k					TOTAL =		49 k-ft	=	50 k-ft

RESISTING MOMENTS, M_r (STR I LOAD COMBO)

	P_v	x	Arm	x	γ_{min} / γ_{max}	=	Min. Moment	=	Max. Moment
Earth:	14,500	x	6.75	x	1.00 / 1.35	=	98 k-ft	=	132 k-ft
Earth:	607	x	8.42	x	1.00 / 1.35	=	5 k-ft	=	7 k-ft
Earth:	0	x	0.00	x	1.00 / 1.35	=	0 k-ft	=	0 k-ft
Earth:	425	x	0.25	x	0.00 / 1.35	=	0 k-ft	=	0 k-ft
Earth:	1,329	x	11.75	x	0.90 / 1.50	=	14 k-ft	=	23 k-ft
Stem:	0	x	0.00	x	0.90 / 1.25	=	0 k-ft	=	0 k-ft
Stem:	2,175	x	1.13	x	0.90 / 1.25	=	2 k-ft	=	3 k-ft
Footing:	5,288	x	5.88	x	0.90 / 1.25	=	28 k-ft	=	39 k-ft
Buoyancy:	-2,278	x	5.71	x	1.00 / 0.00	=	-13 k-ft	=	0 k-ft
Surcharge:	2,500	x	6.75	x	0.00 / 1.75	=	0 k-ft	=	30 k-ft
P_{v_min} = 20.7 k					TOTAL =		134 k-ft	=	234 k-ft
P_{v_max} = 36.7 k									

STRENGTH LIMIT STATE CHECKS:

Overturning

$(1/4)\,L > e = L/2 - (M_r - M_o)/P_v \rightarrow$ **2.94 ft > 1.79 ft** **OK, e WITHIN MIDDLE 1/2**

Sliding Resistance

$P_a < 0.8 \times P_v \times f + 0.5 \times P_p \rightarrow$ **8.3 k < 8.3 k** **OK**

Bearing Resistance

If foundation support by SOIL: Max Bearing Pressure = Min Bearing Pressure = $P_v / (L - 2 \times e)$

If foundation supported by ROCK and 'e' within middle 1/3 of total footing length:

Max Bearing Pressure = $(P_v / L) \times (1 + 6 \times e / L)$ Min Bearing Pressure = $(P_v / L) \times (1 - 6 \times e / L)$

If foundation supported by ROCK and 'e' outside middle 1/3 of total footing length:

Max Bearing Pressure = $(2 \times P_v) / (3 \times (L/2 - e))$ Min Bearing Pressure = 0

$e = L/2 - (M_r - M_o)/P_v = 0.85$ ft

Max Bearing Pressure = **3,648 psf < 4,200 psf** **OK**
Min Bearing Pressure = **3,648 psf < 4,200 psf** **OK**

SHEAR DESIGN:

Stems and footings behave like slabs with one-way shear action. Consequently, the provisions of AASHTO 5.8.2.4 do not apply and the provisions of AASHTO 5.8.3 can be applied in accordance with 5.13.3.6.2.

Design for Stem Shear

AASHTO Eqns. 5.8.3.3-1 & -2

$V_{u\ stem} < \Phi_v \times (V_c + V_s + V_p) \leq \Phi_v \times 0.25 \times (f'_c \times b_v \times d_v + V_p)$

where:

$V_{u\ stem}$ = 5.9 k (passive soil pressure on stem ignored)

Φ_v = 0.90 AASHTO 5.5.4.2.1

V_c = resistance of concrete = $0.0316 \times \sqrt{(f'_c)} \times b_v \times d_v \times \beta$, (AASHTO Eqn. 5.8.3.3-3)
= 16.2 k

V_s = resistance of transverse reinforcement = 0

V_p = vertical component of prestressing force = 0

f'_c = 3.5 ksi

b_v = unit width = 12.0 in

d_v = effective shear depth = max($0.9 \times d_e$, $0.72 \times h$), (AASHTO 5.8.2.9)
= 11.4 in

β = 2.0 (simplified procedure of AASHTO 5.8.3.4.1 assumed)

d_e = depth to tension reinf. = h - 2" clr. - d_b / 2 = 12.7 in

h = stem thickness at base = 15.0 in

d_b = diameter of tensile reinforcement = 0.625 in

$\Phi_v V_n$ = 14.6 k ≤ 107.9 k = 14.6 k

CHECK:

$\Phi_v V_n \geq V_{u\ stem}$

14.6 k ≥ 5.9 k **OK**

Design for Heel Shear

where all values are the same as the stem case, except:

$V_{u\ heel}$ = 32.4 k (soil pressure under heel is ignored per AASHTO 11.6.1.2)

V_c = 38.4 k

f'_c = 3.0 ksi

d_v = max($0.9 \times d_e$, $0.72 \times h$) = 29.3 in

d_e = depth to tension reinf. = h - 3" clr. - d_b / 2 = 32.5 in

h = thickness of footing = 36.0 in

d_b = diameter of tensile reinforcement = 1.000 in (max of toe or heel used)

$\Phi_v V_n$ = 34.6 k ≤ 236.9 k = 34.6 k

CHECK:

$\Phi_v V_n \geq V_{u\ heel}$

34.6 k ≥ 32.4 k **OK**

Design for Toe Shear

where all values are the same as the heel case, except:

$V_{u\ toe}$ = 0.0 k (vertical soil above toe conservatively ignored)

CHECK:

$\Phi_v V_n \geq V_{u\ toe}$

34.6 k ≥ 0.0 k **OK**

Design for Key Shear

where all values are the same as the heel case, except:

$V_{u\ key}$ = 0.0 k

Determine minimum key width, h:

h = max (d_v / 0.72 , 24 in) = 24 in

$d_v \geq V_{u\ key}$ / min [$\Phi_v \times 0.0316 \times \sqrt{(f'_c)} \times b_v \times \beta$, $\Phi_v \times 0.25 \times f'_c \times b_v$] = 0.00 in

MOMENT DESIGN:

Flexural Resistance of a Rectangular Section

AASHTO Eqn. 5.7.3.2.2-1

$$\Phi_f M_n = \Phi_f \times A_s \times f_y \times (d - a / 2)$$

where:

$\Phi_f = 0.90$ AASHTO 5.5.4.2.1, assumes tension-controlled section

A_s = area of tension reinforcement (to be calculated)

f_y = yield strength of tension reinforcement = 60.0 ksi

d = depth to centroid of tension reinforcement

$a = c \times \beta_1$, (AASHTO 5.7.3.2.3)

$\beta_1 = 0.85$ (AASHTO 5.7.2.2)

$c = A_s \times f_y / (0.85 \times f'_c \times \beta_1 \times b)$

$f'_c = 3.0$ ksi

b = width of rectangular section = 12.0 in (unit width for analysis)

$$\Phi_f M_n = \Phi_f \times A_s \times f_y \times (d - A_s \times f_y / (1.70 \times f'_c \times b))$$

Determination of Design Moment, M_{des}

AASHTO 5.7.3.3.2

$$M_{des} = \text{MAX}\begin{cases} M_u \\ \text{MIN}\begin{cases} 1.33\, M_u \\ 1.2\, M_{cr} \end{cases} \end{cases}$$

where:

$M_{cr} = S_c \times f_r$ (AASHTO Eqn. 5.7.3.3.2-1)

$S_c = b \times h^2 / 6$

$f_r = 0.37 \times \sqrt{(f'_c)}$, (AASHTO 5.4.2.6)

h = total depth of rectangular section

Determination of Tension Reinforcement, A_s

$$A_s \geq [B - (B^2 - 4 \times C)^{1/2}]/2$$

where:

$B = 1.70 \times b \times f'_c \times d / f_y$

$C = 1.70 \times b \times f'_c \times M_{des} / (\Phi_f \times f_y^2)$

Design for Stem Moment

All design values are as defined above, except:

β_1 = 0.85
f'_c = 3.5 ksi
d = depth to tension reinf. = h - 2" clr. - d_b / 2 = 12.7 in
h = stem thickness at base = 15.0 in
d_b = diameter of tensile reinforcement = 0.625 in
c = 1.226 in

Determine M_{des}:

$$M_{des} = \text{MAX}\begin{cases} M_{u\ stem} = 28.2\ \text{k-ft} \quad \text{(passive soil pressure on stem ignored)} \\ \text{MIN}\begin{cases} 1.33\ M_u = 37.5\ \text{k-ft} \\ 1.2\ M_{cr} = 374\ \text{k-in} \quad = 31.1\ \text{k-ft} \end{cases} \end{cases}$$

M_{des} = 31.1 k-ft

Determine A_s:

$A_s \geq [B - (B^2 - 4 \times C)^{1/2}]/2$
$A_s \geq$ 0.57 in2/ft

where:
B = 15.1
C = 8.2

Bar Size = # 5
Bar Spacing = 6 in
$A_{s\ tot}$ = 0.62 in2/ft

Use #5's spaced at 6 in

Check Tension-Controlled Assumption:
AASHTO Eqn. 5.5.4.2.1-2

$$\Phi_f = \text{MAX}\begin{cases} 0.75 \\ \text{MIN}\begin{cases} 0.90 \\ 0.65+0.15 \times (d/c-1) = \quad 2.05 \end{cases} \end{cases}$$

Φ_f = 0.90 **OK**

Design for Heel Moment (Top Steel)

All design values are as defined above, except:

β_1 = 0.85
f'_c = 3.0 ksi
d = depth to tension reinf. = h - 3" clr. - d_b / 2 = 32.5 in
h = footing thickness = 36.0 in
d_b = diameter of tensile reinforcement = 1.000 in
c = 3.124 in

Determine M_{des}:

$$M_{des} = \text{MAX}\begin{cases} M_{u\ heel} = 173.3\text{ k-ft} \\ \text{MIN}\begin{cases} 1.33\ M_u = 230.5\text{ k-ft} \\ 1.2\ M_{cr} = 1{,}993\text{ k-in} = 166.1\text{ k-ft}\end{cases}\end{cases}$$

M_{des} = 173.3 k-ft

Determine A_s:

$A_s \geq [B - (B^2 - 4 \times C)^{1/2}]/2$
$A_s \geq$ 1.23 in2/ft

where:
B = 33.2
C = 39.3

Bar Size = # 8
Bar Spacing = 7 in
$A_{s\ tot}$ = 1.35 in2/ft

Use #8's spaced at 7 in

Use # 8 @ 6"

Check Tension-Controlled Assumption:
AASHTO Eqn. 5.5.4.2.1-2

$$\Phi_f = \text{MAX}\begin{cases} 0.75 \\ \text{MIN}\begin{cases} 0.90 \\ 0.65+0.15 \times (d/c-1) = 2.06\end{cases}\end{cases}$$

Φ_f = 0.90 **OK**

Design for Toe Moment (Bottom Steel)

All design values are as defined above, except:

$\beta_1 = 0.85$
$f'_c = 3.0$ ksi
d = depth to tension reinf. = h - 3" clr. - d_b / 2 = 32.7 in
h = footing thickness = 36.0 in
d_b = diameter of tensile reinforcement = 0.625 in
$c = 0.477$ in

Determine M_{des}:

$$M_{des} = \text{MAX} \begin{cases} M_{u\,toe} = 0.4 \text{ k-ft} \\ \text{MIN} \begin{cases} 1.33\, M_u = 0.5 \text{ k-ft} \\ 1.2\, M_{cr} = 1{,}993 \text{ k-in} \quad = 166.1 \text{ k-ft} \end{cases} \end{cases}$$

$M_{des} = 0.5$ k-ft

Determine A_s:

$A_s \geq [B - (B^2 - 4 \times C)^{1/2}]/2$
$A_s \geq 0.20$ in2/ft

where:
B = 33.3
C = 0.1

Bar Size = # 5
Bar Spacing = 18 in
$A_{s\,tot} = 0.21$ in2/ft

Use #5's spaced at 18 in

Use # 5's @ 12" per IDM Fig. 62-3D

Check Tension-Controlled Assumption:
AASHTO Eqn. 5.5.4.2.1-2

$$\Phi_f = \text{MAX} \begin{cases} 0.75 \\ \text{MIN} \begin{cases} 0.90 \\ 0.65+0.15 \times (d/c-1) = \quad 10.78 \end{cases} \end{cases}$$

$\Phi_f = 0.90$ **OK**

Required Temperature & Shrinkage Reinforcement

Temperature & shrinkage requirement does not apply to footings (refer to requirements outlined in AASHTO 5.10.8).

AASHTO 5.10.8

$$A_s \geq \text{MAX} \begin{cases} 0.11 \text{ in}^2\text{/ft E.F.} \\ \text{MIN} \begin{cases} 0.60 \text{ in}^2\text{/ft E.F.} \\ 1.30 \times b \times h / [\,2 \times (b + h) \times F_y\,] \end{cases} \end{cases}$$

where:

b = unit width = 12 in
h = component thickness (in)
F_y = 60.00 ksi ≤ 75.00 ksi

Determine A_s for Stem:

$A_s \geq$ 0.11 in2/ft E.F.

where: h= 15.0 in

Bar Size = # 5 (usually limited to #4's or #5's
Bar Spacing = 18
$A_{s\ tot}$ = 0.21 in2/ft E.F.

Use #5's spaced at 18 in, E.F.

Use # 5's @ 12" per IDM Fig. 62-3D

Lap Length of Stem Tension Reinforcement

Is stem reinforcement epoxy-coated? NO

Tension Lap Splice = CLASS C

AASHTO 5.11.5.3.1, 5.11.2.1.1 & 2

$$\text{Lap Length} = \lambda_s \times l_d = \text{MAX} \begin{cases} \lambda_s \times \lambda_e \times 1.25 \times A_b \times f_y / \sqrt{(f'_c)} \\ \lambda_s \times \lambda_e \times 0.4 \times d_b \times f_y \\ 12.0 \text{ in} \end{cases}$$

where:

l_d = total development length of the bars, in.
λ_s = modification factor for splice class = 1.7
λ_e = modification factor for epoxy-coated bars = 1
A_b = area of the reinf. bar = 0.31 in2
d_b = diameter of the bar = 0.625 in
f_y = 60.0 ksi
f'_c = 3.5 ksi

Lap Length = 25.50 in

ANALYSIS SUMMARY:

Soil and Stability = **All Checks OK**
Shear Design = **All Checks OK**
Moment Design = **All Checks OK**

11.5.5—Load Combinations and Load Factors

Abutments, piers and retaining structures and their foundations and other supporting elements shall be proportioned for all applicable load combinations specified in Article 3.4.1.

C11.5.5

Figures C11.5.5-1 and C11.5.5-2 show the typical application of load factors to produce the total extreme factored force effect for external stability of retaining walls. Where live load surcharge is applicable, the factored surcharge force is generally included over the backfill immediately above the wall only for evaluation of foundation bearing resistance and structure design, as shown in Figure C11.5.5-3. The live load surcharge is not included over the backfill for evaluation of eccentricity, sliding or other failure mechanisms for which such surcharge would represent added resistance to failure. Likewise, the live load on a bridge abutment is included only for evaluation of foundation bearing resistance and structure design. The load factor for live load surcharge is the same for both vertical and horizontal load effects.

The permanent and transient loads and forces shown in the figures include, but are not limited to:

- Permanent Loads

  DC = dead load of structural components and nonstructural attachments
  DW = dead load of wearing surfaces and utilities
  EH = horizontal earth pressure load
  ES = earth surcharge load
  EV = vertical pressure from dead load of earth fill

- Transient Loads

  LS = live load surcharge
  WA = water load and stream pressure

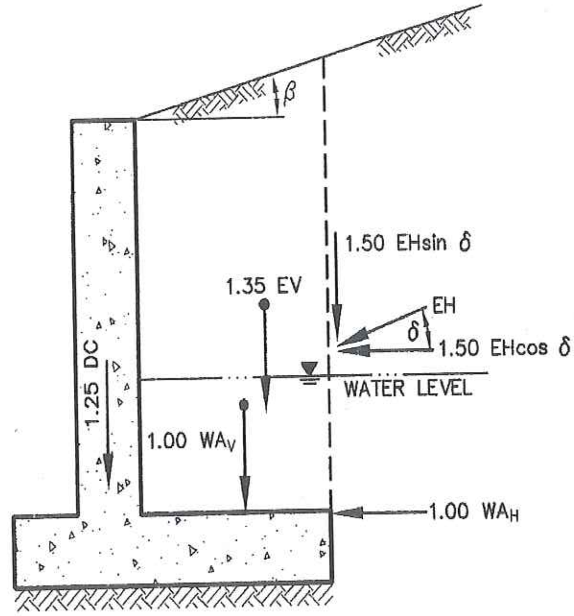


Figure C11.5.5-1—Typical Application of Load Factors for Bearing Resistance

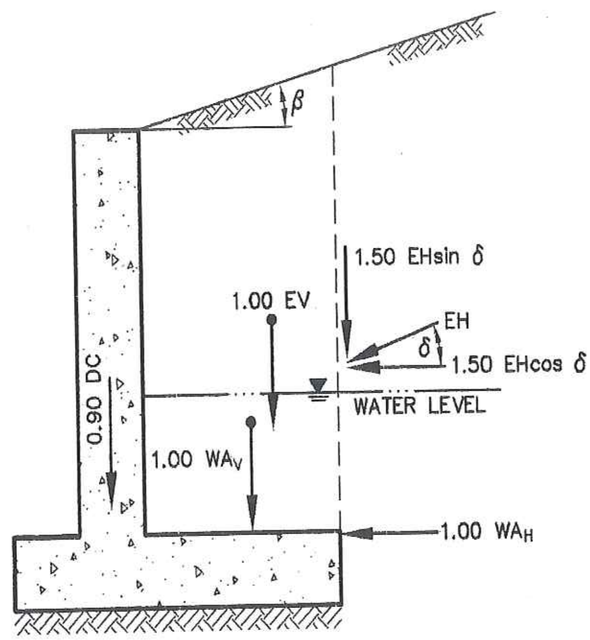


Figure C11.5.5-2—Typical Application of Load Factors for Sliding and Eccentricity

distributed over an area deemed suitable for the size of the structure and the anticipated impacting vehicle, but not greater than 5.0 ft wide by 2.0 ft high. These dimensions were determined by considering the size of a truck frame.

For the purpose of this Article, a barrier may be considered structurally independent if it does not transmit loads to the bridge.

Full-scale crash tests have shown that some vehicles have a greater tendency to lean over or partially cross over a 42.0-in. high barrier than a 54.0-in. high barrier. This behavior would allow a significant collision of the vehicle with the component being protected if the component is located within a few ft of the barrier. If the component is more than about 10.0 ft behind the barrier, the difference between the two barrier heights is no longer important.

Where the design choice is to redirect or absorb the collision load, protection shall consist of one of the following:

- An embankment;
- A structurally independent, crashworthy ground-mounted 54.0-in. high barrier, located within 10.0 ft from the component being protected; or
- A 42.0-in. high barrier located at more than 10.0 ft from the component being protected.

Such barrier shall be structurally and geometrically capable of surviving the crash test for Test Level 5, as specified in Section 13.

3.6.5.2—Vehicle Collision with Barriers

The provisions of Section 13 shall apply.

3.7—WATER LOADS: *WA*

3.7.1—Static Pressure

Static pressure of water shall be assumed to act perpendicular to the surface that is retaining the water. Pressure shall be calculated as the product of height of water above the point of consideration and the specific weight of water.

Design water levels for various limit states shall be as specified and/or approved by the Owner.

3.7.2—Buoyancy

Buoyancy shall be considered to be an uplift force, taken as the sum of the vertical components of static pressures, as specified in Article 3.7.1, acting on all components below design water level.

C3.7.2

For substructures with cavities in which the presence or absence of water cannot be ascertained, the condition producing the least favorable force effect should be chosen.

3.7.3—Stream Pressure

3.7.3.1—Longitudinal

The pressure of flowing water acting in the longitudinal direction of substructures shall be taken as:

$$p = \frac{C_D V^2}{1{,}000} \qquad (3.7.3.1\text{-}1)$$

where:

p = pressure of flowing water (ksf)

C3.7.3.1

For the purpose of this Article, the longitudinal direction refers to the major axis of a substructure unit.

The theoretically correct expression for Eq. 3.7.3.1-1 is:

$$p = C_D \frac{w}{2g} V^2 \qquad (C3.7.3.1\text{-}1)$$

where:

ATTACHMENT NO. 3

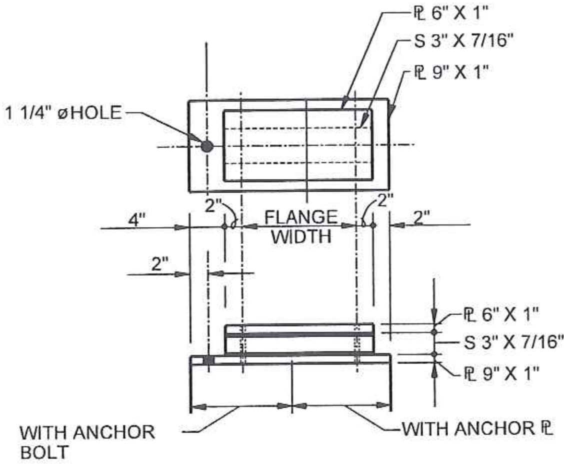
CONSTRUCTION LOADING

The exterior girder has been checked for strength, deflection, and overturning using the constructions loads shown below. Cantilever overhang brackets were assumed for support of the deck overhang past the edge of the exterior girder. The finishing machine was assumed to be supported 6 in. outside the vertical coping form. The top overhang brackets were assumed to be located 6 in. past the edge of the vertical coping form. The bottom overhang brackets were assumed to be braced against the intersection of the girder bottom flange and web.

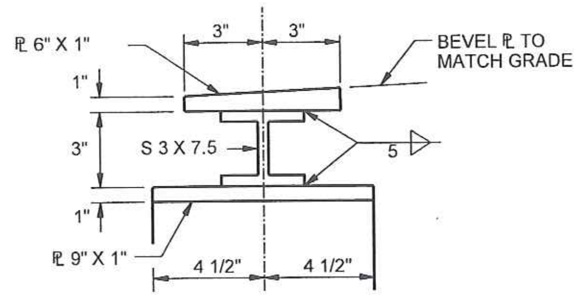
Deck Falsework Loads:	Designed for 15 lb/ft^2 for permanent metal stay-in-place deck forms, removable deck forms, and 2-ft exterior walkway.
Construction Live Load:	Designed for 20 lb/ft^2 extending 2 ft past the edge of coping and 75 lb/ft vertical force applied at a distance of 6 in. outside the face of coping over a 30-ft length of the deck centered with the finishing machine.
Finishing-Machine Load:	4500 lb distributed over 10 ft along the coping.
Wind Load:	Designed for 70 mph horizontal wind loading ~~of 50 lb/ft^2~~ 25 in accordance with AASHTO *Guide Design Specifications for Bridge Temporary Works* (1995), Figure 2.1. The wind load shall be calculated per AASHTO LRFD 3.8.1.

CONSTRUCTION-LOADINGS INFORMATION TO BE SHOWN ON GENERAL PLAN

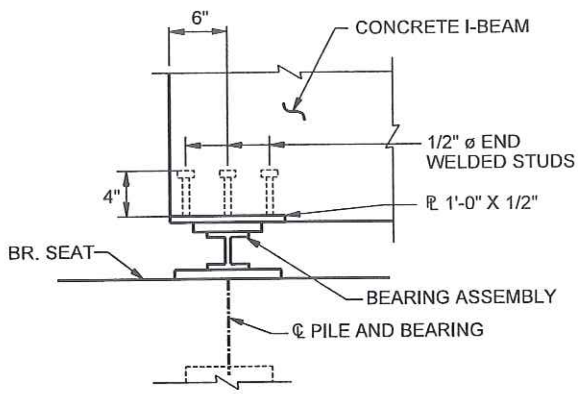
Figure 403-4A



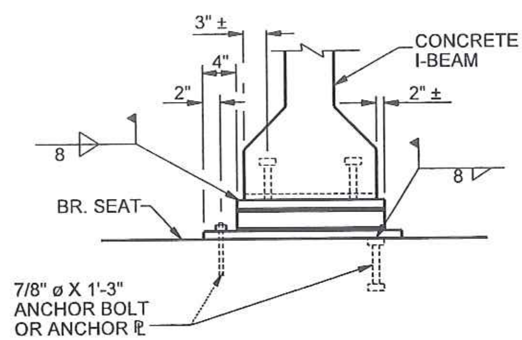
BEARING ASSEMBLY
TOP / SIDE VIEW



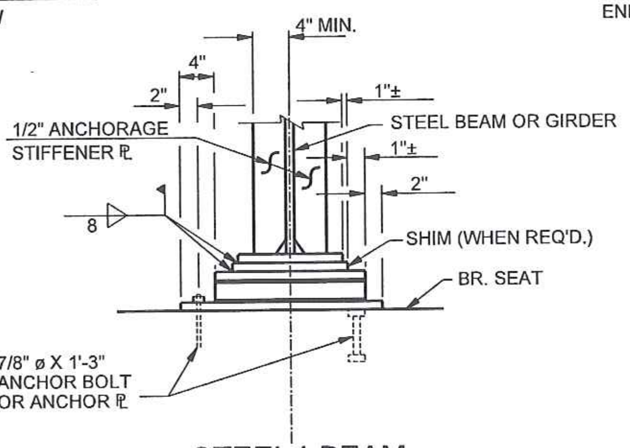
BEARING ASSEMBLY
END VIEW



CONCRETE I-BEAM
SIDE VIEW



CONCRETE I-BEAM
END VIEW



STEEL I-BEAM
END VIEW

SUGGESTED INTEGRAL END BENT DETAILS
(Beams Attached Directly to Concrete Cap, Method B)

Figure 409-2C
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