

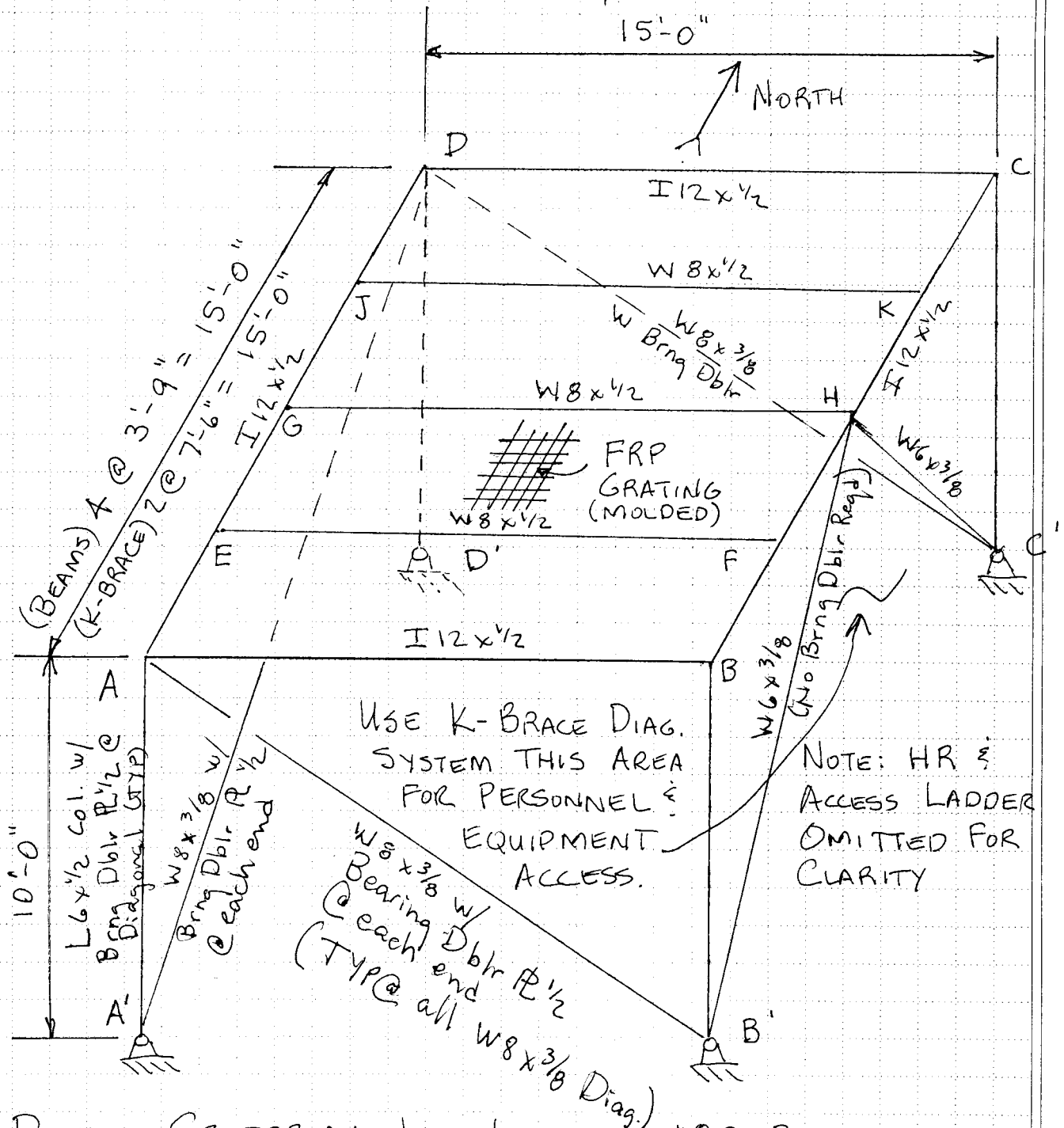
DESIGN EXAMPLE

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DESIGN CRITERIA: LIVE LOAD OF 100 PSF

OPERATING WIND OF 60 MPH

ENVIRONMENT: OFFSHORE, STORM WIND OF 166 MPH

OPERATING TEMPERATURE 125° F

DEFLECTION $\leq L/150$ (BEAMS/GIRDERS)

GRATING DEFLECTION $\leq 0.25"$

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SIZE GRATING

SPECIFICATION REQUIRES THE USE OF A MOLDED GRATING.

$$\text{SPAN} = 3'-9" (45")$$

FROM DELTAGRATE MOLDED GRATING BROCHURE, TRY 1 1/2" THK X 1 1/2" SQ. MESH.

FROM CHART FOR 100 PSF @ 42", $\Delta = .26"$

$$\text{FOR } 45" \text{ SPAN, } \Delta = .26 \left(\frac{45^4}{42^4} \right) = 0.34" > .25" \text{ /NG}$$

TRY 2" THK X 2" SQ. MESH

FROM TABLE, @ 100 PSF @ 42" SPAN, $\Delta = 0.111$

$$\text{FOR } 45" \text{ SPAN, } \Delta = .111 \left(\frac{45^4}{42^4} \right) = 0.15" < 1/4" \text{ /OK}$$

Use 2" Thk x 2" SQUARE MESH
MOLDED GRATING



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BEAM SIZING

SIZE MEMBERS EF, GH, JK (SPAN = 15'; LL = 100 PSF)

FRP BEAMS ARE TYPICALLY GOVERNED & SIZED ON THE BASIS OF MEETING DEFLECTION REQUIREMENTS, THEN STRESSES ARE CHECKED.

USE CREATIVE SUPERSTRUCTURALS 1525 SERIES

EFFECTS OF TEMPERATURE - FROM SECTION 7 OF DESIGN MANUAL, REDUCE ULTIMATE STRESSES BY 30% AND MODULI BY 10% @ 125°F TEMP.

TRY W 8 x 1/2

$$\begin{cases} I_x = 128.81 \text{ in}^4 \\ E = 3.9 \times 10^6 \text{ psi} \times 0.9 = 3.51 \times 10^6 \text{ psi} \\ A_w = (8)(1/2) = 4 \text{ in}^2 \\ G = 0.5 \times 10^6 \text{ psi} \times 0.9 = .45 \times 10^6 \end{cases}$$

CHECK DEFLECTION

$$\Delta = \frac{5 w L^4}{384 E I} + \frac{w L^2}{8 A' G}$$

Flexural deformation

shear deformation

Effective beam width

$$(A' = K A_w)$$

From Table 2, K = 1.0

$$W = \underbrace{(100 \text{ PSF}) 3.75 \text{ FT}}_{\text{LIVE LOAD}} + \underbrace{(4 \text{ PSF}) 3.75' + 10 \frac{\text{lb}}{\text{ft}}}_{\text{DEAD Wt}}$$

GRATING Wt BEAM Wt

$$W = 400 \text{ lb/ft} = 33.33 \text{ lb/in}$$

$$\Delta = \frac{(5)(33.33)(15)^4(12)^4}{(384)(3.9 E6)(.9)(128.81)} + \frac{(33.33)(15)^2(12)^2}{(8)(4)(.5 E6)(.9)} = 1.07''$$

$$\Delta/L = \frac{1.07}{(15)(12)} = 1/167 < 1/150 \text{ OK}$$

W 8 x 1/2 OK for deflection
Check Stresses

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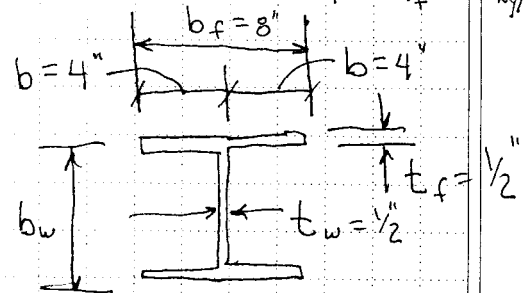
BEAM SIZING (Cont'd)

CHECK STRESSES (W 8x1/2)

FIRST CHECK LOCAL FLG BUCKLING (From Pg 51)

$$\sigma_{xcr} = \frac{\pi^2}{12} \left(\frac{t_f}{b} \right)^2 \left[\sqrt{q} \left(2 \sqrt{(E_{c-LW})_f (E_{c-cw})_f} + p \left((E_{c-cw})_f (v_{LW})_f + 2 G_{xyf} \right) \right) \right]$$

 $b = \frac{b_f}{2}$ for I & W Shapes

 $b = b_f$ for Box Sections


$$p = .3 + \left[\frac{.004}{\zeta - .5} \right] = 0.3 + \left[\frac{.004}{(2 - .5)} \right] = .30267$$

$$q = .025 + \left[\frac{.065}{\zeta + .4} \right] = .025 + \left[\frac{.065}{(2 + .4)} \right] = .05208$$

$$\zeta = \frac{2 b_w \cdot (E_{c-cw})_f}{b_f \cdot (E_{c-cw})_w} = \frac{(2)(8)(1.9 \times 10^6)}{(8)(1.9 \times 10^6)} = 2$$

$$b = \frac{b_f}{2} \text{ for I & W Shapes} = \frac{8}{2} = 4$$

$$(E_{c-LW})_f = 3.85 \times 10^6 \text{ psi}$$

$$(E_{c-cw})_f = 1.9 \times 10^6 \text{ psi}$$

$$(E_{c-cw})_w = 1.9 \times 10^6 \text{ psi}$$

Thus

$$\sigma_{xcr} = \left(\frac{\pi^2}{12} \right) \left(\frac{.5}{4} \right)^2 \left[\sqrt{.052} \left(2 \sqrt{(3.85 \times 10^6)(1.9 \times 10^6)} \right) + .30267 \left((1.9 \times 10^6)(.35) + 2(.5 \times 10^6) \right) \right]$$

$$\sigma_{xcr} = 22,327.98 \text{ psi}$$



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BEAM SIZING (CONT'D)

CHECK STRESSES (CONT'D)

LOCAL BUCKLING CHECK (CONT'D)

ACCOUNTING FOR THE EFFECTS OF TEMPERATURE,
WE MUST LOWER σ_{cr} BY THE 30% (SEE CAPT. 7)

$$\text{THUS } \sigma'_{cr} = (0.7) \sigma_{cr} = 15,629.6 \text{ psi}$$

$$\text{Flexure S.F.} = 2.5 \Rightarrow F_{b \text{ LOCAL BUCKLING}} = 6,251.8 \text{ psi}$$

$$\text{MAX } M_x = \frac{wL^2}{8} = \frac{(33.33)(15 \cdot 12)^2}{8} = 134,986.5 \text{ in-lbs}$$

$$\text{S' fo } W 8 \times 1/2 = 32.2 \text{ in}^3 \Rightarrow f_b = \frac{M_x}{S_x} = 4,192.13 \text{ psi}$$

$$f_{bx} < F_{b \text{ LOCAL BUCKLING}} \quad / \text{OK}$$

CHECK GLOBAL BUCKLING

$$w = 33.33 \text{ lb/in}$$

$$L = 15' (180")$$

Try unbraced length of compression flange = $L_b = 180"$

$$P_{cr}^{\text{global}} = \frac{17.17}{L_u^2} \sqrt{D \cdot JG_{xy}} \sqrt{1 + \frac{\pi^2}{L_u^2} \frac{I_{ww}}{JG_{xy}}}$$

L_u = Unbraced Length of Compression flange = 180"

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BEAM SIZING (CONT'D)

CHECK STRESSES (CONT'D)

GLOBAL BUCKLING CHECK (CONT'D)

$$JG = \frac{2(G_{xy})_f t_f^3 b_f}{3} + \frac{(G_{xy})_w t_w^3 b_w}{3}$$

$$(G_{xy})_f = (G_{xy})_w = .5 \times 10^6 \text{ psi}$$

$$t_f = t_w = \frac{1}{2} ; b_f = b_w = 8"$$

$$JG = \frac{2(.5E6)(\frac{1}{2})^3(8)}{3} + \frac{(.5E6)(\frac{1}{2})^3(8)}{3} = 500,000 \text{ in}^4 - \frac{\text{lb}}{\text{in}^2}$$

D = EI of THIN-WALLED LAMINATED BEAMS
USING MECHANICS OF LAMINATED BEAMS THEORY

$$D = \frac{1}{2}(E_{c-lw})_f t_f b_w^2 b_f + \frac{1}{12}(E_{c-lw})_w t_w b_w^3 + \frac{1}{6}(E_{c-lw})_f t_f^3 b_f$$

$$(E_{c-lw})_f = 3.85 \times 10^6 \text{ psi} ; t_f = t_w = \frac{1}{2}$$

$$(E_{c-lw})_w = 2.8 \times 10^6 \text{ psi} ; b_w = b_f = 8"$$

$$D = \left(\frac{1}{2}\right)(3.85E6)\left(\frac{1}{2}\right)(8)^2(8) + \left(\frac{1}{12}\right)(2.8E6)\left(\frac{1}{2}\right)(8)^3 + \frac{1}{6}(3.85E6)\left(\frac{1}{2}\right)^3(8)$$

$$= 553,175,000 \text{ in}^4 - \frac{\text{lb}}{\text{in}^2}$$

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BEAM SIZING

CHECK STRESSES (CONT'D)

GLOBAL BUCKLING CHECK (CONT'D)

$$I_{ww} = \frac{(E_c - L_w)_f t_f b_w^2 b_f^3}{24} + \frac{(E_c - L_w)_f t_f^3 b_f^3}{36} + \frac{(E_c - L_w)_w t_w^3 b_w^3}{144}$$

$$I_{ww} = \frac{(3.85E6)(\frac{1}{2})(8)^2(8)^3}{24} + \frac{(3.85E6)(\frac{1}{2})^3(8)^3}{36} + \frac{(2.8E6)(\frac{1}{2})^3(8)^3}{144}$$

$$I_{ww} = 2,636,355,556 \text{ lb-in}^4$$

$$\text{Thus } P_{cr}^{\text{global}} = \left(\frac{17.17}{180^2} \right) \sqrt{(553,175,000)(500,000)} \sqrt{1 + \frac{\pi^2}{180^2} \cdot \frac{2,636,355,556}{500,000}}$$

$$= 14,227.9 \text{ lbs}$$

$$\text{Thus } M_{cr}^{\text{global}} = \frac{P_{cr}^{\text{global}} \cdot L}{4} \Rightarrow \sigma_{cr}^{\text{global}} = \frac{P_{cr}^{\text{global}} \cdot L}{4 S}$$

$$\sigma_{cr}^{\text{global}} = \frac{(14,227.9)(180)}{4(32.20)} = 19,883.8 \text{ psi}$$

↑
S_x for W8 x 1/2

Accounting for the effects of Temperature @ 125°F:

$$\sigma_{cr}'^{\text{global}} = 0.7(\sigma_{cr}^{\text{global}}) = 13,918.64 \text{ psi}$$

$$\text{Bending S.F.} = 2.5, \therefore F_b^{\text{global}} = 5,567.4 \text{ psi}$$



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BEAM SIZING (CONT'D)
 CHECK STRESSES (CONT'D)
 GLOBAL BUCKLING CHECK (CONT'D)

$$f_b = \frac{M_x}{S_x} = \frac{134,986.5 \text{ in-lb}}{32.2 \text{ in}^3} = 4192.3$$

$$f_b < F_b^{\text{global}} / \text{OK}$$

Thus, no additional lateral bracing of the
 W8x12 compression flg is needed

Note: Had the F_b^{global} been less than the
 acting bending stress, f_b , we would have
 had to go back into the calculation of
 P_{cr}^{global} , used a different value of
 L_u , plugged it into the equation &
 ultimately arrive at an $F_b^{\text{global}} \geq f_b$.



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GIRDER SIZING
SIZE MEMBER AD

$$L = 15'$$

$$W = \underbrace{(100 \text{ PSF} \times 7.5')}_{LL} + \underbrace{(7 \text{ PSF} \times 7.5') + 10 \text{ lb/ft}}_{DL}$$

BEAM + Grtg Wt Beam Wt

$$W = 812.5 \text{ lb/ft}$$

$$M = \frac{WL^2}{8} = 274,218.8 \text{ in-lbs}$$

SIZE FOR DEFLECTION

$$\Delta = \frac{5WL^4}{384EI T_m} + \frac{WL^2}{8A'_w G T_m} \leq L/150$$

$$@ T = 125^\circ \text{F}, T_m = 0.9 \text{ (EFFECTS OF TEMP.)}$$

$$\Delta = \frac{(5) \left(\frac{812.5}{12} \right) (180)^4}{(384)(3.9E6)(I)(.9)} + \frac{\left(\frac{812.5}{12} \right) (180)^2}{(8)(A'_w)(.5E6)(.9)}$$

$$\Delta = \frac{263.67}{I} + \frac{.609375}{A'_w}$$

$$A'_w = A_w \sqrt{k} \quad k=1$$

$$A'_w = A_w$$

TRY I 12 x 1/2

$$I = 258.76$$

$$A_w = (12)(1/2) = 6$$

$$\Delta = 1.12''$$

$$\Delta/L = \frac{1.12}{180} = \frac{1}{160.6} < \frac{1}{150} / \text{OK}$$



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GIRDER SIZING

MEMBER AD (Cont'd)

Check Local Buckling

I 12 x 1/2

LOCAL BUCKLING $S_x = 43.13 \text{ in}^3$

$$(E_{c-cw})_f = 1.9 \text{ E}6 \text{ psi} \quad (E_{c-lw})_f = 3.85 \text{ E}6 \text{ psi}$$

$$(E_{c-cw})_w = 1.9 \text{ E}6 \text{ psi} \quad (E_{c-lw})_w = 2.8 \text{ E}6 \text{ psi}$$

$$\nu_{xyf} = \nu_{xyw} = 0.35 \quad b_w = 12" \quad b_f = 6"$$

$$t_f = t_w = 1/2 \quad G_{xy} = .5 \text{ E}6$$

$$b = b_f/2 = 3" \quad T_s = 0.7 @ 125^\circ \text{F}$$

$$Q = \frac{2(12)(E_{c-cw})_f}{(6)(E_{c-cw})_w} = 4$$

$$q = .025 + \left(\frac{.065}{4.4} \right) = .03977$$

$$p = .3 + \left(\frac{.004}{4 - .5} \right) = .3011$$

$$\begin{aligned} \sigma_{cr}^{Local} &= \frac{T_s \pi^2}{12} \left(\frac{t_f}{b} \right)^2 \left[\sqrt{q} \left(2 \sqrt{(E_{c-lw})_f (E_{c-cw})_f} + p \left((E_{c-cw})_f (\nu_{xy})_f + 2(G_{xy})_f \right) \right) \right] \\ &= \frac{(.7) \pi^2}{12} \left(\frac{1/2}{3} \right)^2 \left[\sqrt{.03977} \left(2 \sqrt{(3.85 \text{ E}6)(1.9 \text{ E}6)} + .3011 \left((1.9 \text{ E}6)(.35) + 2(.5 \text{ E}6) \right) \right) \right] \\ &= 18,850.46 \text{ psi} \end{aligned}$$

$$F_{b_{local}} = \frac{\sigma_{cr}^{Local}}{FS} = 7540.2 \text{ psi}$$

FS $\rightarrow$ 2.5

$$M_{allow} = F_{b_{local}} \cdot S_x = 325,208.6 \text{ in-lbs} > 274,219 \text{ OK}$$



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GIRDER SIZING
MEMBER AD
GLOBAL BUCKLING

$$P_{cr}^{global} = \frac{17.17}{(L_u)^2} \sqrt{D \cdot JG} \sqrt{1 + \frac{\pi^2}{(L_u)^2} \frac{I_{ww}}{JG}} \cdot T_s$$

L_u = Unbraced length of compression flg. = 3.75' (45")

$$\begin{aligned} JG &= \frac{2(G_{xy})_f t_f^3 b_f}{3} + \frac{(G_{xy})_w t_w^3 b_w}{3} = \\ &= \frac{2(.5E6)(\frac{1}{2})^3(6)}{3} + \frac{(.5E6)(\frac{1}{2})^3(12)}{3} \\ &= 500,000 \text{ lb-in}^2 \end{aligned}$$

$$\begin{aligned} D &= \frac{1}{2}(E_{c-lw})_f t_f b_w^2 b_f + \frac{1}{12}(E_{c-lw})_w t_w b_w^3 + \frac{1}{6}(E_{c-lw})_f t_f^3 b_f \\ &= \frac{1}{2}(3.85E6)(\frac{1}{2})(12)^2(6) + \frac{1}{12}(3.85E6)(\frac{1}{2})(12)^3 + \frac{1}{6}(3.85E6)(\frac{1}{2})^3(6) \\ &= 1,109,281,250 \text{ lb-in}^2 \end{aligned}$$

$$\begin{aligned} I_{ww} &= 3.85E6 \left[\frac{(\frac{1}{2})(12)^2(6)^3}{24} + \frac{(\frac{1}{2})^3(6)^3}{36} + \frac{(\frac{1}{2})^3(12)^3}{144} \right] \quad (\text{Eqn 9-12}) \\ &= 2,503,462,500 \text{ lb-in}^4 \end{aligned}$$



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Girder Sizing
Member AD (Cont'd)
Global Buckling (Cont'd)

$$P_{cr}^{global} = \frac{17.17}{(45)^2} \sqrt{D \cdot JG} \sqrt{1 + \frac{\pi^2}{45^2} \frac{I_{ww}}{JG}} \cdot (0.7)$$

$L_u \uparrow$ $\uparrow T_s$

$$= 704,519 \text{ lbs}$$

$$\sigma_{cr}^{global} = \frac{P_{cr}^{global}}{4 S_x} = \frac{(704,519)(15 \times 12)}{(4)(43.13)} = 735065 \text{ psi}$$

Global buckling is not a problem since we are braced every 45"

Use I 12 x 1/2 for Girder AD



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COLUMN SIZING

Effective Area each column = $7.5' \times 7.5' = 56.25 \text{ ft}^2$

$$LL = 100 \text{ PSF}$$

$$DL = 7 \text{ PSF}$$

$$107 \text{ PSF} \times 56.25 \text{ ft}^2 = P = 6020 \text{ \#}$$

$$L = 10'$$

$$K = 1.0$$

Try L 6 x 1/2

$$A = 5.72 \text{ in}^2$$

$$r_z = 1.177$$

$$\frac{KL}{r_z} = \frac{(1)(120)}{1.177} = 101.95$$

$$E_{c-LW} = 3.0 \times 10^6 \text{ psi}$$

Local Buckling Check

$$\sigma_{cr-local} = \Phi k \frac{1}{12} \left(\frac{E_{c-LW}}{1 - \nu^2} \right) \left(\frac{t_f}{b} \right)^2 T_s$$

$$= (0.8)(0.5) \left(\frac{1}{12} \right) \left(\frac{3E6}{(1 - 0.35^2)} \right) \left(\frac{1/2}{6} \right)^2 (0.7) = 3,324 \text{ psi}$$

Global Buckling

$$\sigma_{ult,Euler} = \frac{\pi^2 (E_{c-LW})}{\left(\frac{KL}{r} \right)^2} T_s = \frac{\pi^2 (3E6)(0.7)}{(101.95)^2} = 1994 \text{ psi}$$

Governs ✓



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Column Sizing (Cont'd)

For L-Sections, you must also check flexural-torsional buckling $\sigma_{ult, f-t}$

$$\sigma_{ult, f-t} = \Phi \frac{E_c - \nu w}{2(1 + \nu)} \left(\frac{t_f}{b_f} \right)^2 \cdot T_s$$

$$b = b_f = 6$$

$$\sigma_{ult, f-t} = (.8) \frac{(3.0 \text{ E}6)}{2(1 + .35)} \left(\frac{1/2}{6} \right)^2 \cdot .7 = 4,320 \text{ psi}$$

Global Buckling controls

$$F_{a-allow} = \frac{\sigma_{ult, Euler}}{S.F.} = \frac{1994}{3.0} = 664 \text{ psi}$$

↖ 3.0

$$P_{allow} = F_{a-allow} \times A = \frac{(664)}{(5.72)} = 3801 \text{ \#} < 6020 \text{ \#}$$

~~NG~~

L6 x 1/2 Columns
NO GOOD!

Use W6 x 3/8 (Calcs omitted)



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WIND AREA

BASED UPON PRELIMINARY CALCS, COLUMNS ARE 6" WIDE,
ASSUME DIAGONALS ARE W6'S

CALCULATE WIND AREA NORTH

Assumptions

1. Assume every 6th Grating bearing bar fully effective
2. Assume HP Posts every 4 ft

$$A_{\text{Grts}} = 16 \text{ bars} \times \left(\frac{2}{12}\right)' (15') = 40 \text{ ft}^2$$

$$A_{\text{HR POST}} = \left(\frac{2}{12}\right) (3.5') \left(\frac{60}{4}\right) = 9 \text{ ft}^2$$

$$A_{\text{HR RAIL/KICKER}} = \left(\frac{8}{12}\right) 15' = 11 \text{ ft}^2$$

$$A_{\text{I12}} = (1) (15') (2) = 30 \text{ ft}^2$$

$$A_{\text{W8}} = \left(\frac{8}{12}\right) (3) (15') = 30 \text{ ft}^2$$

$$A_{\text{COL}} = \frac{1}{2} \left(\frac{6}{12}\right) (5') (4) = 5 \text{ ft}^2$$

$$A_{\text{DIAG}_{\text{II}}} = \left(\frac{1}{2}\right) \left(\frac{6}{12}\right) (5') (3) = 4 \text{ ft}^2$$

$$A_{\text{DIAG}_{\perp}} = \left(\frac{1}{2}\right) \sqrt{10^2 + 15^2} \left(\frac{6}{12}\right) (2) = 9 \text{ ft}^2$$

$$138 \text{ ft}^2$$

$$\times 1.10 \text{ MISC}$$

$$152 \text{ ft}^2$$

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WIND Forces

$$P = .00256 V^2 C_s A$$

Shape Factor = 1.5 for flat surfaces
= 1.0 for cylinders

Operating Wind Speed = 60 mph

Storm Wind Speed $\Rightarrow$ 166 mph

$$P_{oper} = 2102 \#$$

Total Wind forces on structure

$$P_{storm} = 16,084 \#$$

$$P_{col. Row} = \frac{P}{(\# \text{ of Col. Rows})} = \frac{P}{2}$$

$$P_{col. Row OPER} = 1051 \#$$

$$P_{col. Row STORM} = 8042 \#$$

SIZE DIAGONAL $\overline{A'D}$ **Delta Composites, L.L.C.**

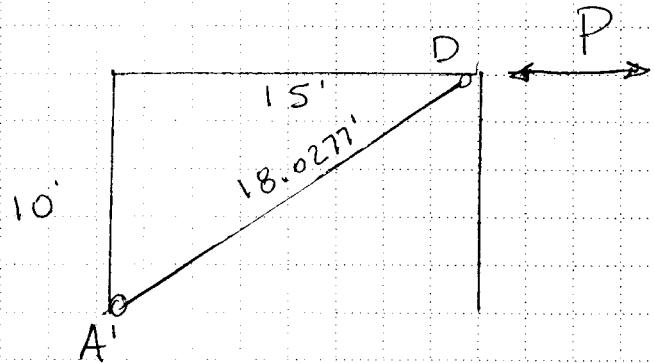
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DIAGONAL SIZING
MEMBER A'D

Analyze Storm Condition First

$$P = 8042 \text{ \#}$$

$$P_{A'D} = 8042 \left(\frac{18.0277}{15} \right) = 9665.3 \text{ \#}$$

$$K = 1.0$$

Try W6 x 14 $A = 4.42 \text{ in}^2$ $r_y = 1.42$ $b_f = 6"$, $b = 3$
 $E_{c-LW} = 3.85 \times 10^6 \text{ psi}$ $\nu = .35$

$$\frac{KL}{r_y} = \frac{(1)(18.0277)(12)}{1.42} = 152.36$$

Check Short Column

$$\sigma_{cr-local} = \Phi_k \frac{\pi^2 (E_{c-LW})}{12(1-\nu^2)} \left(\frac{t_f}{b} \right)^2 = (.8)(.5) \frac{\pi^2 (3.85E6) \left(\frac{1}{4} \right)^2}{12 (1-.35^2) (3)^2}$$

$$\sigma_{cr-local} = 10,023.7 \text{ psi}$$

$$F_a^{local} = \left(\sigma_{cr-local} / F.S. \right) \times T_s = 10,023.7 \left(\frac{.7}{3.0} \right) = 2338.9 \text{ psi}$$

3.0 for compression

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Diagonal Sizing
Member A'D (cont'd)

Check Global Buckling

$$\sigma_{ult, Euler} = \frac{\pi^2 (E_{c-LW}) I_s}{\left(\frac{KL}{r_y}\right)^2} = \frac{(\pi^4)(3.85 \times 10^6)(.7)}{(152.36)^2}$$

$$= 1636.9 \text{ psi.} \quad \checkmark \text{ governs over local}$$

$$F_{a-allow} = \sigma_{ult, Euler} / F.S. = 545.63 \text{ psi}$$

↑
3.0

Storm Case

$$P_{allow} = 545.63 (4.42) (1.33) = 3215 \# < 9665 \#$$

1/3rd = Increase for Storm Loads

NG

Try W8 x 3/8

$$A = 11.07 \text{ in}^2$$

$$E_{c-LW} = 3.85 \times 10^6$$

$$r_y = 1.90 \text{ in}$$

$$b_s = 8, \quad b = 4 \text{ in}$$

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$$\frac{KL}{r_y} = \frac{(1)(18.0277)(12)}{1.90} = 113.86$$

Check Local Buckling

$$\tau_{cr-local} = (0.8)(0.5) \frac{\pi^2}{12} \frac{(3.85 \text{ E6})}{(1-.35^2)} \frac{(\frac{3}{8})^2}{(4)^2} T_s (0.7) = 8,880 \text{ psi}$$

Check Global Buckling

$$\tau_{ult, Euler} = \frac{\pi^2 (E_{c-lw})}{\left(\frac{KL}{r_y}\right)^2} T_s = 2,051.7 \text{ psi} \checkmark \text{ governs}$$

$$F_{a-allow} = \frac{\tau_{ult, Euler}}{F.S.} = 683.9 \text{ psi}$$

↑ 3.0

Storm Case:

$$P_{allow} = (F_{a-allow})(A) (1.33) = 10,094^\# > 9665^\#$$

↑ 11.07

1/3rd increase permitted for storm cases

OK

Use W8 x 3/8 for Diagonal A'D

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DESIGN EXAMPLE

ENGINEER

PROJECT NO.

DATE

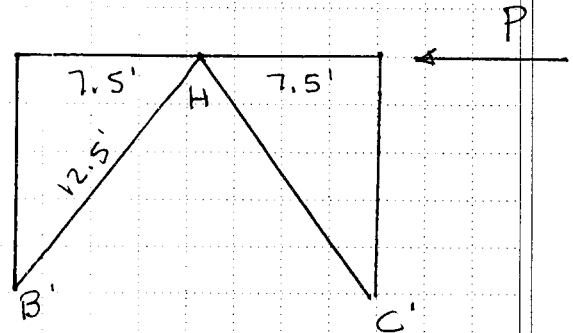
1/04

DIAGONAL SIZING
SIZE DIAGONAL B'H, C'H

STORM CASE

$$P = 8042 \text{ \#}$$

$$B'H = \frac{8042 \text{ \#}}{2} \left(\frac{12.5'}{7.5'} \right) = 6702 \text{ \#}$$



Try W6 x 3/8

$$K = 1.0$$

$$L = 12.5'$$

$$A = 6.57 \text{ in}^2$$

$$r_y = 1.42 \text{ in}$$

$$b_f = 6'; b = 3"$$

$$\frac{KL}{r_y} = \frac{(1)(12.5)(12)}{1.42} = 105.63$$

Local Buckling

$$\sigma_{cr-local} = \Phi_k \frac{\pi^2 (E_{c-lw})}{12 (1 - \nu^2)} \left(\frac{t_f}{b} \right)^2 T_s$$

$$= (0.8)(0.5) \left(\frac{\pi^2}{12} \right) \frac{(3.85 \times 10^6)}{(1 - 0.35^2)} \left(\frac{3/8}{3} \right)^2 (1.7) = 15,787 \text{ psi}$$

$$\sigma_{ult, Euler} = \frac{\pi^2 (E_{c-lw})}{\left(\frac{KL}{r_y} \right)^2} T_s = \frac{\pi^2 (3.85 \times 10^6)}{(105.63)^2} (1.7) = 2,384 \text{ psi}$$

$$P_{a-allow} = \frac{\sigma_{ult}}{S.F.} = 795 \text{ psi}$$

$$P_{allow} = (795) \cdot (6.57) \cdot (1.33) = 6960 \text{ \#} > 6702 \text{ \#} \text{ OK}$$

Use W6 x 3/8 @ Diagonal B'H



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DESIGN EXAMPLE

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CONNECTION DESIGN
DIAG A'D to Col. A'A

W8x $\frac{3}{8}$ Diag to L6x $\frac{1}{2}$ Col.

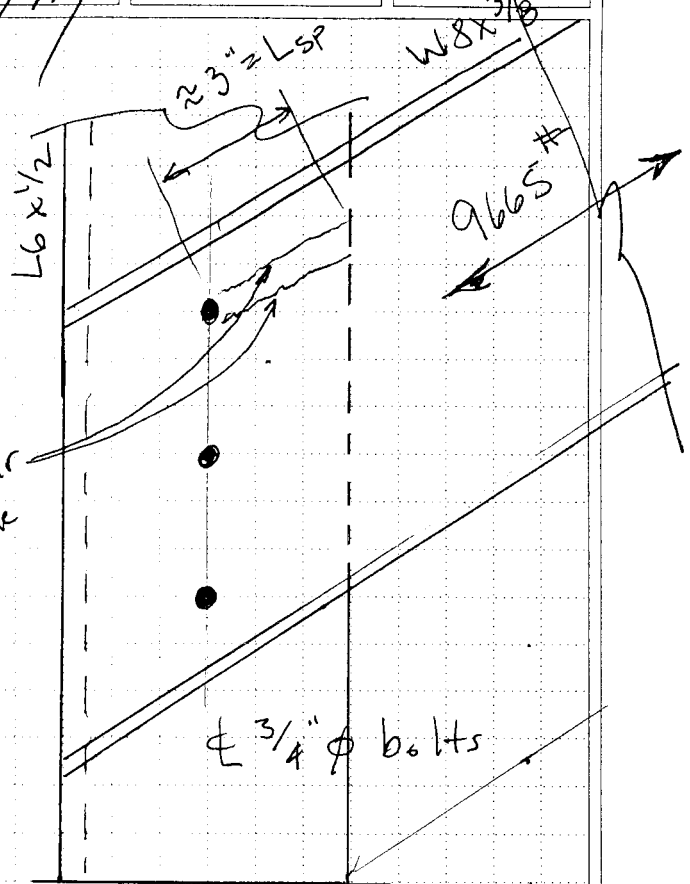
Check Bearing Strength
Using $\frac{3}{4}$ " ϕ bolts

L6x $\frac{1}{2}$ bearing strengths

$$F_{brg-w_{ult}} = 33,000 \text{ ps}$$

$$F_{brg-w_{ult}} = 23,000$$

Shear
Typ. Plane



The bulk of the load is
being introduced to the L6
in the CW direction

$$F_{brg} = \frac{23000 (T_s)^{.7}}{S.F.} = \frac{16,100 \text{ ps}}{4} = 4025 \text{ psi}$$

$\frac{1}{3}^{rd}$ increase in allow stress

$$\begin{aligned} P_{allow_{brg}} &= F_{brg} A_{brg} = (4025) (\text{No. bolts}) \phi_{bolt} t (1.33) \\ &= (4025) (3) \left(\frac{3}{4}\right) \left(\frac{1}{2}\right) \\ &= 6038 \text{ \# NG} \end{aligned}$$

Add Bearing Doubler to increase
thickness, $t_{reqd} = \frac{9665}{6038} \geq 1.6 \left(\frac{1}{2}\right) = .8$ "

To get to .8", add $\frac{1}{2}$ " bearing doubler

total
bearing
thickness
reqd



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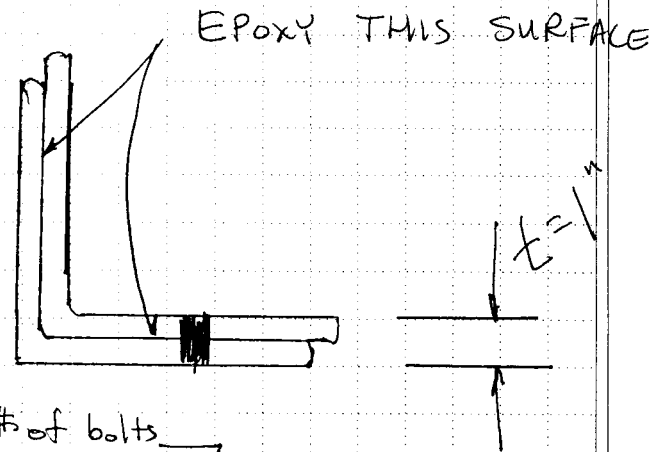
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CONNECTION DESIGN
DIAG A'D to Col A'A (Cont'd)

To achieve this, epoxy another L6 x 1/2 to the
L6 x 1/2 Col as shown below



$$P_{allow-brg} = (F_{brg}) (A_{brg}) = (4025) (3) \left(\frac{3}{4}\right) (1) 1.33$$

of bolts bolt ϕ t

$$= 12,045^{\#} > 9665^{\#} \quad \text{OK}$$

Check Shear Capacity of L6 Joint
From Sect 4

$$F_{v-cw-ult} = 4500 \text{ psi}$$

$$F_v = F_{v-cw-ult} \cdot T_s = 4500(.7) = 3150 \text{ psi}$$

$$A_s = (\# \text{ of bolts}) (t) (L_{sp}) \left(\frac{\# \text{ of shear planes}}{\text{bolt}} \right)$$

$$= (3)(1)(3)2 = 18 \text{ in}^2$$

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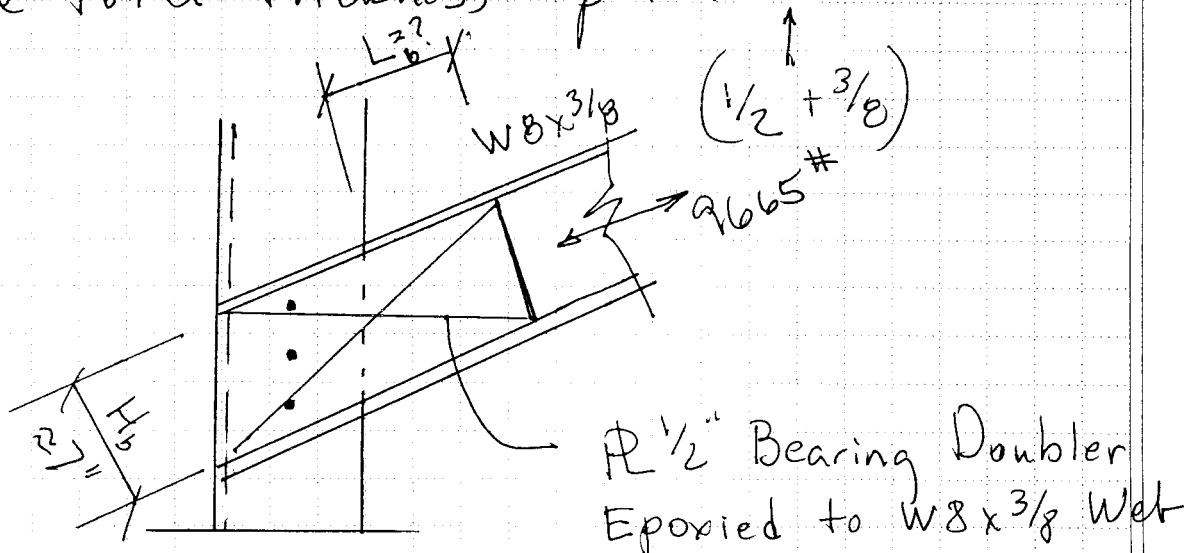
Connection Design
Diag. A'D to Col A'A (Cont'd)

$$\begin{aligned}
 P_{\text{allow-shear}} &= F_v A_s \\
 &= 3150 (18) = 56,700 \text{ \#} > 9,665 \text{ \#} \\
 &\quad \text{OK}
 \end{aligned}$$

Check Web of W8x3/8 Diagonal

From previous calcs, we know that for bearing we need at least 0.8" of thickness.

Add Web Bearing Doubler $\Phi 1/2$ to get the total thickness up to .875"



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Connection Design
Diag A'D to Col A'A (Cont'd)

Because we are epoxying a $\frac{1}{2}$ " Brg Dblr to a $\frac{3}{8}$ " Thk Web, it will carry more load

$$"P." \text{ in doubler} = \left(\frac{\frac{1}{2}}{\frac{3}{8}} \right) 9665 \# = 5523 \#$$

This 5523# must be transferred thru bond area where the doubler has been epoxyed to the web

$$\text{Epoxy Adhesion} = 1000 \text{ psi} \\ \div 4.0 \text{ Safety Factor}$$

$$F_{adh} = 250 \text{ psi}$$

$$A_{adh} = \text{Bond Area} = L_b H_b = L_b 7$$

$$P_{allow} = 7 L_b \cdot F_{adh} = 7 L_b (250) \\ = 1750 L_b = 9665$$

$$L_b = 5.5"$$

Use at least 7"



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CONNECTION DESIGN
DIAG B'H TO GIRDER BC (CONT'D)

$$F_{t-cw} = \frac{10,500}{\text{S.F.}} \left(\frac{T_s}{4.0} \right) (1.33) = 2450 \text{ psi.}$$

Allow stress increase
for storm case

$$P_{allow} = 2450 (6) = 14,700 \text{ \#} > 5362 \text{ \#}$$

OK

Check Bolt Bearing onto Web of Clip-T

From Sect 4, $F_{brg-wlt-cw} = 30,000 \text{ psi.}$

$$F_{brg} = \frac{30,000}{\text{S.F.}} \left(\frac{T_s}{4.0} \right) (1.33) = 6983 \text{ psi.}$$

of bolts

$$A_{brg} = \frac{3}{4} 3 \left(\frac{1}{2} \right) = 1.125 \text{ in}^2$$

bolt ϕ t

$$P_{allow-brg} = 1.125 (6983) = 7855 \text{ \#} > 6702 \text{ \#}$$

OK

No Need to Add a Bearing Doubler

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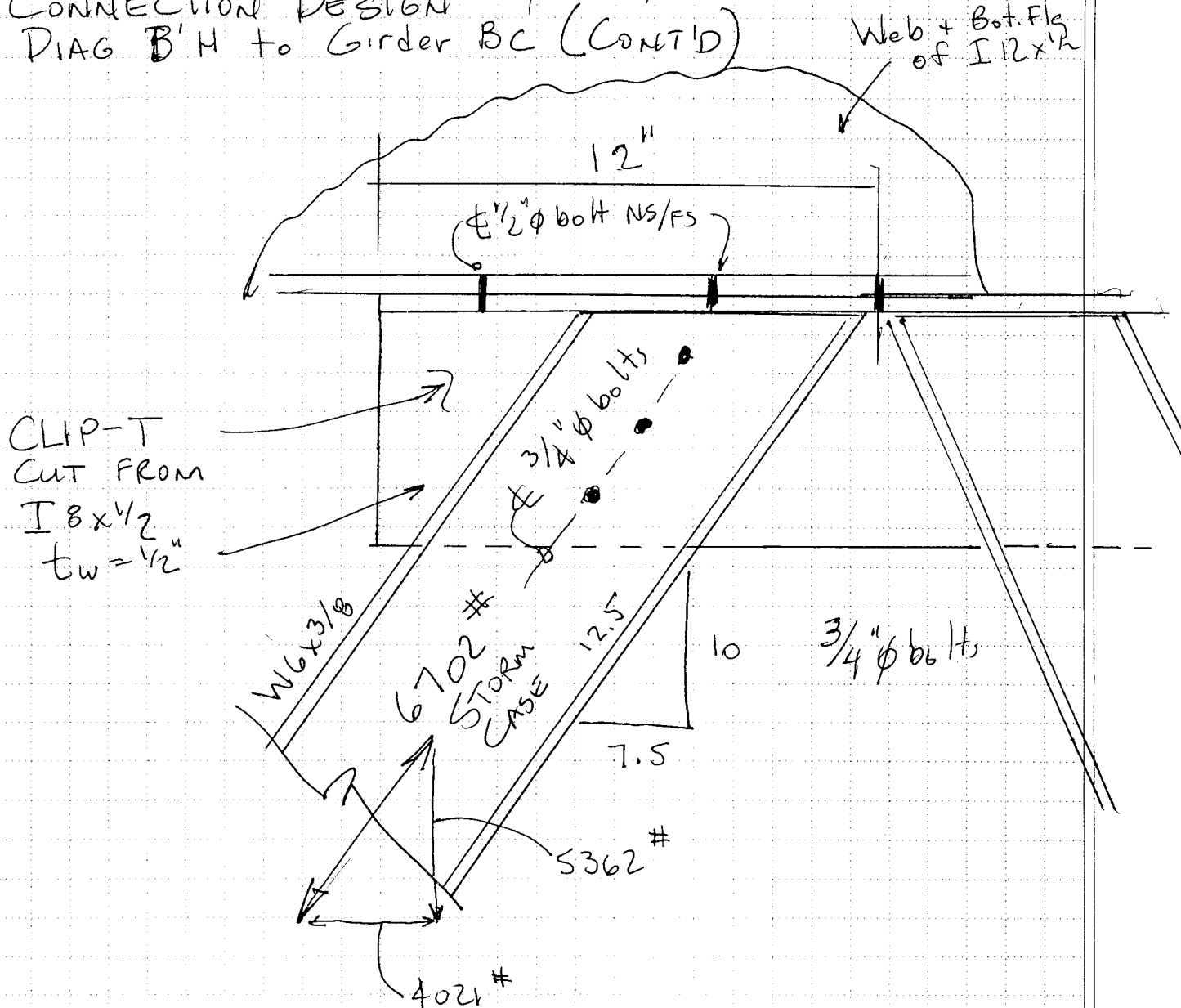
ENGINEER

RYM

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CONNECTION DESIGN
DIAG B'H to Girder BC' (CONT'D)

Check Capacity of Clip-T to resist the normal force of $5362 \#$

The clip T will mobilize CW Tensile resistance

$$A_{clipT} = 12(1/2) = 6 \text{ in}^2$$

From Sect. 4, Web Tensile Strength $CW = 10,500$



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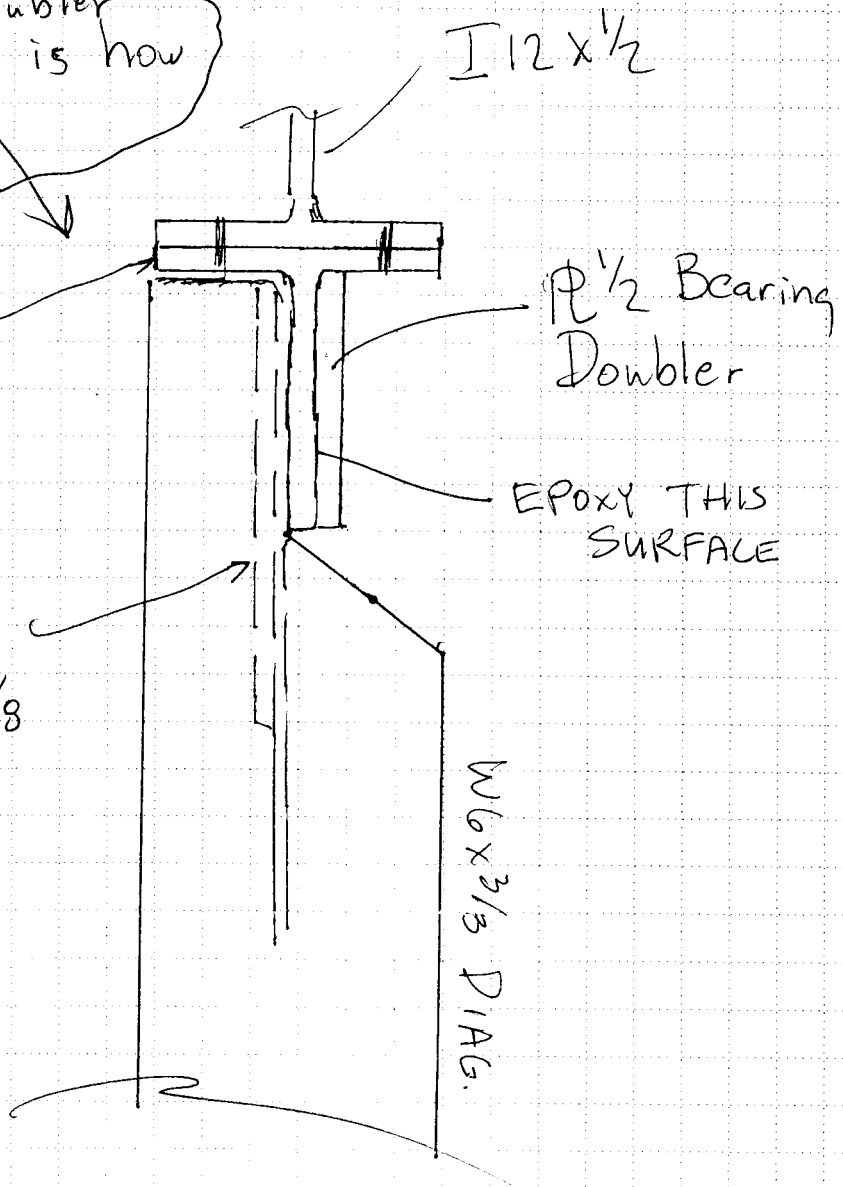
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CONNECTION DESIGN
DIAG. B'H TO GIRDER BC (CONT'D)

If a bearing doubler were req'd, this is how it would be accomplished

CLIP-T
from W8x1/2

Add Bearing
Doubler to
Web W6x3/8

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CONNECTION DESIGN
DIAG B'H TO GIRDER BC (CONT'D)Analyze $\frac{1}{2}" \phi$ bolts bearing on Flg @ $\frac{1}{2}$ of I12 $\frac{3}{4}$ WB
Clip-T

$$F_{brg-wlt-Lw} = 33,000$$

$$F_{brg-Lw} = \frac{33,000}{S.F.} \cdot T_s = \frac{33,000}{4.0} (.7) = 5775 \text{ psi}$$

$$A_b = (\# \text{ of bolts}) (\phi) (t_f)$$

$$= (4) \left(\frac{1}{2}\right) \left(\frac{1}{2}\right) = 1 \text{ in}^2$$

$$P_{allow-brg} = F_{brg-Lw} \cdot A_b = 5775 (1) = 5775^{\#} > 4021^{\#}$$

OK

Use of $\frac{1}{2}" \phi$ attachment bolts
w/o flg bearing @s OK



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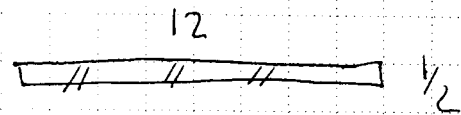
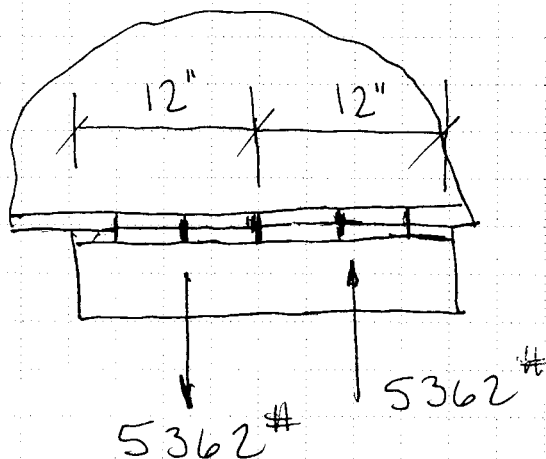
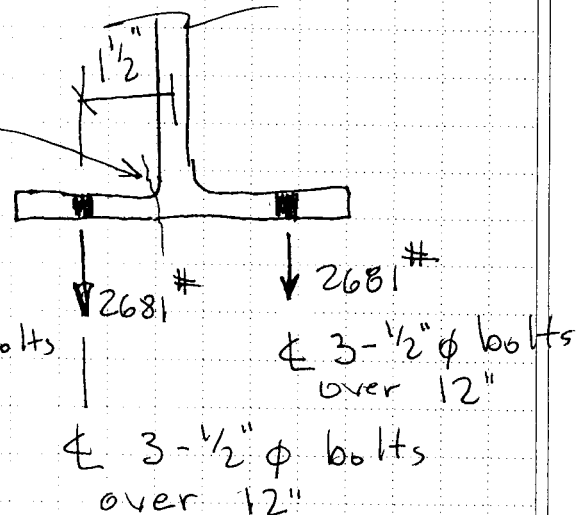
1/04

CONNECTION DESIGN
DIAG. B'H TO GIRDER BC (CONT'D)

CHECK THE FLG OF I12 to resist the weak axis bending due to 5362# force normal to the flange

Calc. stress @
this line

$$2681\# = \frac{5362\#}{2 \text{ rows of bolts}}$$



$$S' = \frac{bd^2}{6} = .5 \text{ in}^3$$

$$M = 2681 (1.5") = 4021.5 \text{ in-lbs} \quad f_b = \frac{M}{S} = 8043 \text{ psi}$$

From Chpt 4, Flg Flex. Strength $CW = 17,360 \text{ psi}$

$$F_b = \frac{17360}{(4.0)} (1.7) (1.33) = 4051 \text{ psi} < 8043 \text{ psi}$$

S.F.

NG



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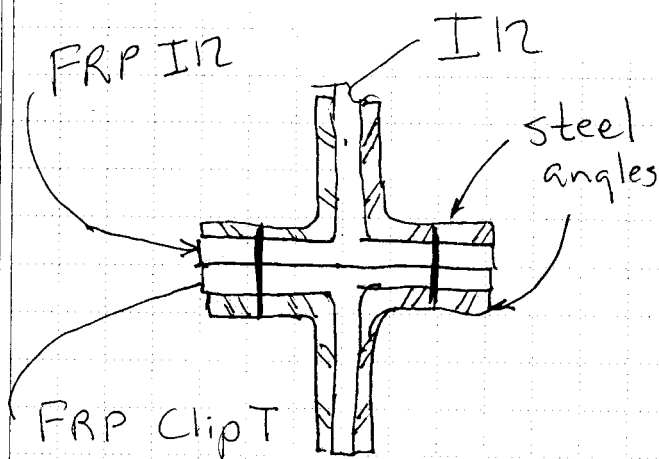
PROJECT NO.

DATE

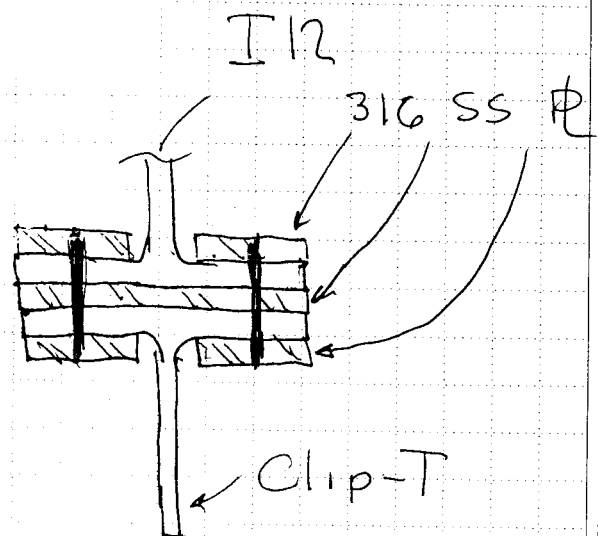
4/04

CONNECTION DESIGN
DIAG B'M TO GIRDER BC (CONT'D)

THE FLG is bending out of plane & is failing due to the 5362 # load normal to the flange — i.e., the tension is peeling the flange off. Add some additional angles (steel or 316 SS) as shown below, or sandwich in a flat PL



Add Angles



Sandwich Plate

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