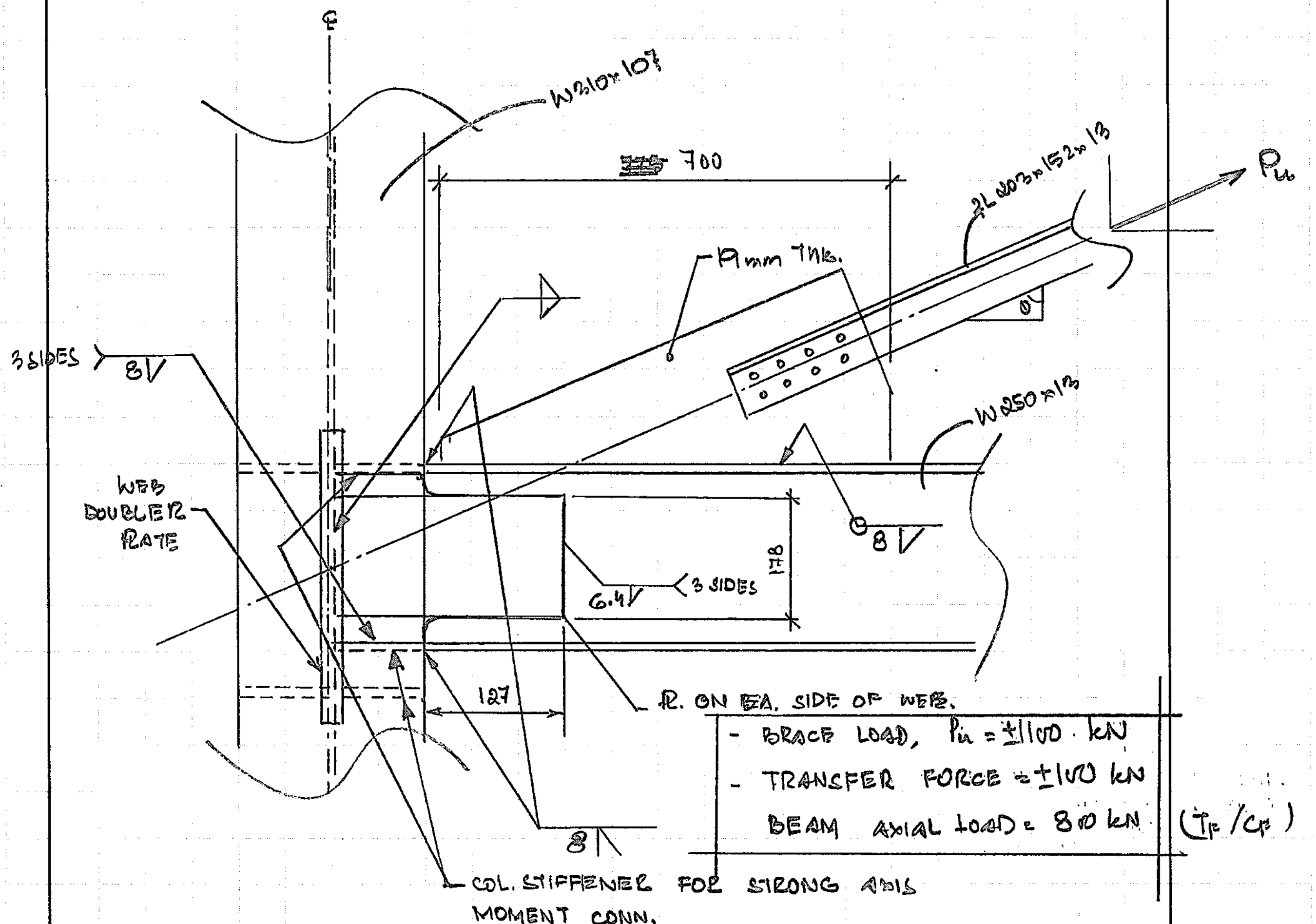
armtec 100TH ANNIVERSARY 1908-2008

PROJECT ____________ NUM. ________

DATE ________

BY ____________ SHEET ______ OF ______



COLUMN: W310x107 ⇒ $d = 311$; $b_c = 305$; $t = 17.0$; $w = 10.9$; $k = 39.7$, [illegible]

BEAM : W250x73 ⇒ $d_b = 254$; $b = 254$; $t = 14.20$; $w = 8.64$, $k_1 = 20.6$

BRACE: 2L 203x152x13 (LONG LEG DOWN)

CONNECTION INTERFACE FORCES :-

$e_b = d_b/2 = 127$ mm

$e_c = b_c/2 = 152.5$ mm

$\tan\theta = 65^\circ$

NO CONN. TO COLUMN ⇒ $\beta = 0$

$\therefore \alpha = e_b \tan\theta - e_c \cong 120$ mm

$\bar{\alpha} = 700/2 + 0 = $ ~~[illegible]~~ 350 mm

A. GUSSET TO BEAM :

$H_{ub} = H = P_u \cdot \sin\theta = 997$ kN

$V_{ub} = V = P_u \cdot \cos\theta = 465$ kN

$M_{ub} = V_{ub}(\bar{\alpha} - \alpha) = 107$ kNm

B. BEAM TO COLUMN :

$M_{bc} = V_{ub} \cdot e_c = 70.9$ kNm

$V_{bc} = V_{ub} + R_u = 465$ kN

$A_u = 900$ kN

A. DESIGN(GUSSET TO BEAM)

1. Gusset plate for shear yielding + tensile yielding →.

$f_{ua} = \frac{V_{ub}}{A} = \frac{465 \text{ kN} \times 10^3}{(700 \times 19)} = 35$ MPa

$f_{ub} = \frac{M_{ub}}{Z} = \frac{107 \times 10^6}{(19 \times 700^2/4)} = 46$ MPa

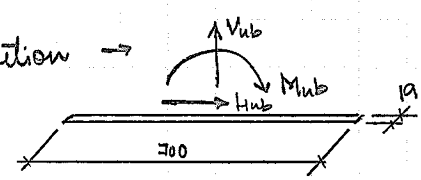
∴ $\Sigma f = f_{ua} + f_{ub} = 80.97$ MPa $< 0.9F_y$ (270) ⇒ ok

Check shear yielding →

$f_{uv} = \frac{997 \times 10^3 \text{ N}}{19 \times 700} = 75$ MPa $< 0.6F_y$ (180) ⇒ ok

2. Design weld @ gusset to beam connection →

Design as weld group :



∴ $S_x = bd + d^2/3$

$= 700 \times 19 + 700^2/3 = 176633.3$ mm^3

AREA, $A = 1438$ mm^2

∴ ↑ $\sigma_M = M_{ub}/S_x \approx 606$ MPa (BENDING)

↑ $\tau_{V_{ub}} = V_{ub}/A = 323$ MPa

→ $\tau_{H_{ub}} = H_{ub}/A = 693$ MPa

∴ Resultant, $R = \left((\sigma_M + \tau_{V_{ub}})^2 + \tau_{H_{ub}}^2\right)^{1/2}$

$= 1159.0$ MPa

∴ Use fillet weld size, $d_w = R/156 = 7.45$ mm

⇒ use $d_w = 8$ mm (5/16") min.

3. Check beam web local yielding :

$N_{equiv.} = V_{ub} + \frac{2 M_{ub}}{(L/2)} = 465 + \frac{2 \times 107 \times 10^3}{350} = 1076.5$ kN ~~mm~~

∴ $B_y = \phi_{bi}\, w\, (N + 10t)\, F_y = 0.80 \times 8.64\, (700 + 10 \times 14.20) \times 0.350$

(Interior) $= \cancel{1745}$ kN $> N_{equiv.}$ (1076.5) ⇒ Ok

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$B_r = \phi_{bc} w (N + 4L) F_y = 1716.4$ kN > Nequiv (1077) → Ok
(End Reaction)

B. DESIGN (BEAM TO COLUMN)

Reqd. shear strength, $V_u = 405$ kN
" Axial " , $A_u = 900$ kN
" Moment " , $M_u = 70.9$ kNm

FLANGE WELDED CONN. DESIGN ⟶

Flange force due to (M_u), $P_{f1} = M_u / d_b = 279$ kN ⟶

" " due to axial load (A_u), $P_{f2} = \dfrac{900 \times 254 \times 14.20}{9290}$

$= 350$ kN ⟶

∴ $\Sigma P_f = P_{f1} + P_{f2} = 628$ kN.

∴ Reqd. weld size $= 628 / (b_b + (b_b - 2k_1)) = 1.35$ kN/mm

∴ Use 8mm PJP weld on flange.

WEB WELDED CONN. DESIGN ⟶

Web axial load due to A_u, $P_{f3} = 900 - 2 \times P_{f2} = 200$ kN ⟶

Shear load, $= 405$ kN

∴ Resultant, $R = 506.2$ kN

∴ Reqd. $D = \dfrac{P}{CL}$

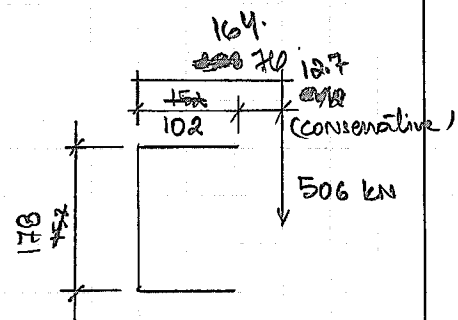
$k = 152/152 = 1.$

∴ $x = 0.333$

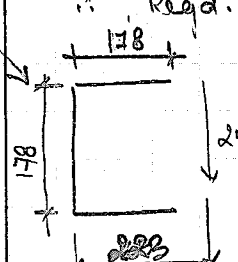
$aL + xL = 228$

∴ $a = \dfrac{228 - 0.333 \times 152}{152} = 1.2$

⇒ $C = 0.214$ (Table 8-28)



∴ Reqd. weld size, $D = \dfrac{506}{0.214 \times 152} = 15.6$ mm ⟶ NO GOOD



⇒ D (Reqd) = 6.4

∴ $V_r = 343.3$ kN > 506/2 (253) kN

⇒ Ok TO USE PL. ON EACH SIDE OF WEB.