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CLIENT ODOT
PROJECT CUY 271
SUBJECT WALL WS2
SHEAR & MOMENT CALCULATIONS (PANEL)

PROJECT NO. 1122-1001-90
SHEET NO. ____ OF ____
COMP. BY ASP DATE 08/26/14
CHECKED BY LNB DATE 08/27/14

SHEAR & MOMENT FOR PANEL DESIGN

SOIL PROPERTIES

ACTUAL FILL MATERIAL PLACED BEHIND WALL SHALL BE GRANULAR MATERIAL, TYPE B

BULK UNIT WEIGHT = 130 PCF

SATURATED " " = 67.6 PCF

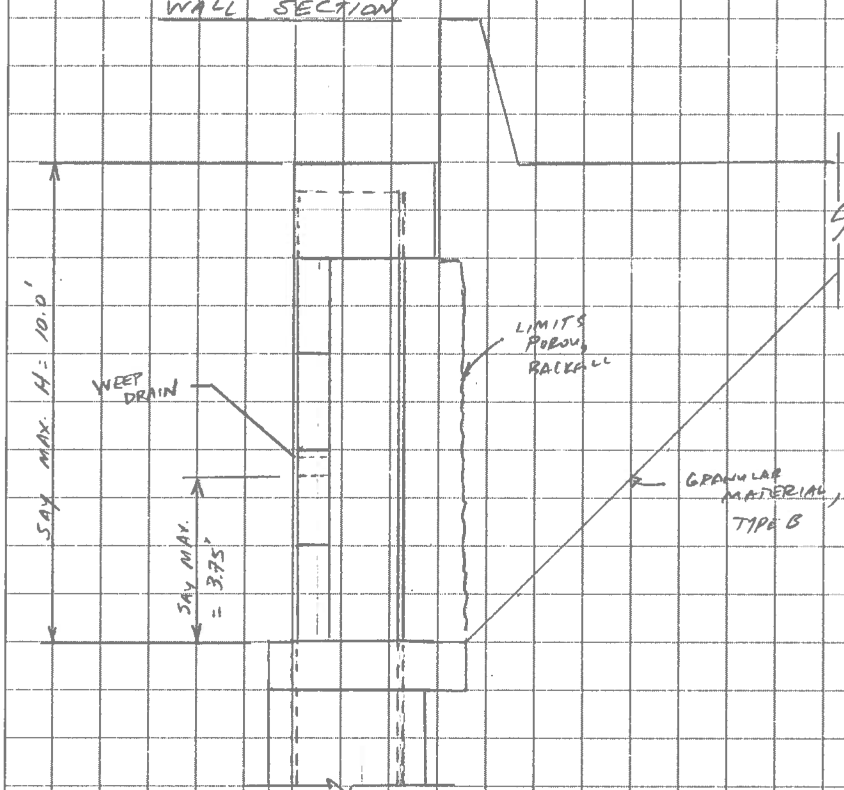
FRICTION ANGLE, ϕ = 34°

AT REST COEFFICIENT, K_o = 0.441

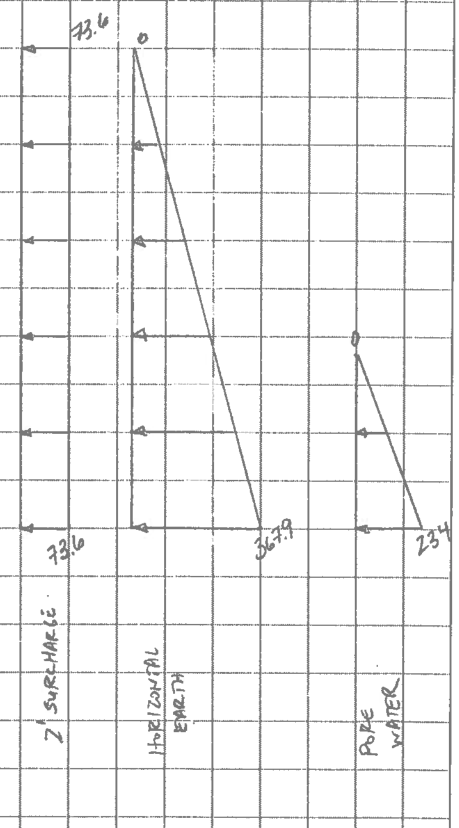
ACTIVE PRESSURE COEFF., K_a = 0.283

PASSIVE PRESSURE ", K_p = 3.537

WALL SECTION



PRESSURE DIAGRAM



2' SURCHARGE → 2' × 0.283 × 130 = 73.6 PSF

HORIZONTAL EARTH → 10' × 0.283 × 130 = 367.9 PSF

PORE WATER → 3.75' × 62.4 PCF = 234 PSF

NOTE: PORE WATER PRESSURE CONSIDER DUE TO LACK OF UNDERDRAIN SYSTEM FOR WALL
PASSIVE PRESSURE IGNORED

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SHEAR AND MOMENT FOR PANEL DESIGN (CONT.)

DESIGN LOADS

-DESIGN ALL PANELS FOR PRESSURE AT BOTTOM OF WALL (CONSERVATIVE)

- MAX. PANEL WIDTH = 6.0' - 2(1.5") = 5.75'

SERVICE SHEAR

$\frac{5.75'}{2} \times (73.6\ PSF + 367.9\ PSF + 234\ PSF) = 1{,}942.1$ P/FT HT.

1.94 K/FT HEIGHT

FACTORED SHEAR

$5.75'/2 \times [1.75(73.6) + 1.50(367.9) + 1.0(234)] = 2{,}629.6$ #/FT HEIGHT

2.63 K/FT HEIGHT

SERVICE MOMENT ($wl^2/8$)

$w = 73.6 + 367.9 + 234 = 675.5$

$M_s = 675.5(5.75^2)/8 =$ 2,791.7 #·FT/FT HEIGHT

2.79 K·FT/FT HEIGHT

FACTORED MOMENT

$w = 1.75(73.6) + 1.5(367.9) + 1.0(234) = 914.7$

$M_F = 914.7(5.75)^2/8 =$ 3,780.1 #·FT/FT HEIGHT

3.78 K·FT/FT HEIGHT

Dimensions and Weights for Concrete Design

Concrete weight, w_c = 0.150 kcf
Concrete strength, f_c' = 4.00 ksi
Rebar strength, f_y = 60.00 ksi
Steel mod. of elast., E_s = 29,000 ksi

Panel Thickness, t_p = 0.50 ft = 6.00 in

Service Design Moment, M_{serv} = 2.79 k-ft
Factored Design Moment, M_u = 3.78 k-ft
Service Design Shear, V_{serv} = 1.94 k
Factored Design Shear, V_u = 2.63 k

Design Reinforcement - Moment Equation

Set up the equation to solve for the required steel area:

$$M_u = \phi M_n = \phi A_s f_y (d_s - a/2) \quad = \quad \phi A_s f_y \left(d_s - \frac{A_s f_y}{1.7 f_c' b} \right)$$

$$M_u = 0.90 \times A_s \times 60 \left(ds - \frac{As \times 60}{1.7 \times 4.0 \times 12} \right) \times \left(\frac{1}{12} \right)$$

$$3.309 \quad A_s^2 \quad - \quad 4.50 \quad d_s A_s + M_u = 0$$

Investigate Shear

$V_n = V_c + V_s + V_p$ [5.8.3.3-1]

Recognizing that V_s and V_p are zero

$V_n = V_c$

$V_c = 0.0316\ \beta\ (f_c')^{0.5}\ b_v\ d_v$ [5.8.3.3-3]

The maximum effective shear depth is:

For 2.0 inch clear cover and #4 bars, d_s = 6.00 - 2.00 - (0.500 / 2) = 3.75 in

$d_v = 0.9\ d_e = 0.9\ d_s$ = 0.90 x 3.75 = 3.38 in
$d_v = 0.72\ h = 0.72\ t_p$ = 0.72 x 6.00 = 4.32 in GOVERNS

Follow the General Procedure using the provisions of Appendix B5, as per Section [5.8.3.4.2]:

β = 3.103831 ε_s= 0.00113 *s_{xe}= 3.657423 Max Aggregate Size = 1 in (BDM S5.10.3.1.1)

*(based on single layer reinforcement)

ϕV_c = 0.90 x 0.0316 x 3.10 x (4.0)$^{0.50}$ x 12.00 x 4.32 = 9.15 k

9.15 k > 2.63 k OK

Wall Stem Design - Investigate Strength Limit State

Determine the area of front-face flexural reinforcement necessary to satisfy the design moment:

M_u = 3.78 k-ft

Again use the equation:

$$3.309 \quad A_s^2 \quad - \quad 4.50 \quad d_s A_s + M_u = 0$$

Substituting and solving for A_s, it is found that required A_s = 0.23 in^2/ft

Try: #4 bars at 6.00 in c/c A_s = 0.40 in^2/ft > 0.23 in^2/ft OK

Wall Stem Design - Investigate Service Limit State [5.7.3.4]

Check the crack control equations to ensure that the primary flexural reinforcement is well distributed. The service load bending moment is:

M_{serv} = 2.79 k-ft

Check the modulus of rupture for concrete:

$\phi f_r = \phi\, 0.24\, (f_c')^{0.5}$ = 0.80 x 0.24 x (4.0)$^{0.50}$ = 0.384 ksi

$S = \dfrac{b\, t_p^2}{6} = \dfrac{12.00 \times (6.00)^2}{6}$ = 72.00 in^3

$f_{act} = \dfrac{M_{serv}}{S} = \dfrac{2.79 \times 12}{72.00}$ = 0.465 ksi > 0.384 ksi NG

If modulus of rupture check is "NG", then check the spacing of the reinforcement. First, determine the modular ratio:

$E_c = 33{,}000\, w_c^{1.5}\, (f_c')^{0.5}$ = 33,000 x (0.150)$^{1.5}$ x (4.00)$^{0.5}$ = 3,834 ksi

$n = E_s/E_c$ = 29,000 / 3,834 = 7.56 , use n = 8.00

For 2.0 inch clear cover and #4 bars, d_s = 6.00 - 2.00 - (0.500 / 2) = 3.75 in

Determine the location of the neutral axis:

$0.5\, b\, x^2 = n\, A_s\, (d_s - x)$

$0.5\,(12)\, x^2$ = 8 x 0.40 x (3.75 - x)

solving, x = 1.17 in

Check the spacing of the reinforcement to control cracking:

$y = d_s - x$ = 3.75 - 1.17 = 2.58 in

$I_{cr} = \dfrac{b\, x^3}{3} + n\, A_s\, (d_s - x)^2$

$I_{cr} = \dfrac{12.00 \times (1.17)^3}{3} + 8 \times 0.40 \times (3.75 - 1.17)^2$ = 27.71 in^4

γ_e = 1.00

For 2.0 inch clear cover and #4 bars, d_c = 2.00 + (0.500 / 2) = 2.25 in

$\beta_s = 1.0 + \dfrac{d_c}{0.7\,(t_p - d_c)} = 1.0 + \dfrac{2.25}{0.7 \times (6.00 - 2.25)}$ = 1.86

$f_{ss} = n\, \dfrac{M_{serv}\, y}{I_{cr}} = 8 \times \dfrac{2.79 \times 2.58 \times 12.00}{27.71}$ = 24.92 ksi

$s_{max} = \dfrac{700\, \gamma_e}{\beta_s\, f_{ss}} - 2.0\, d_c = \dfrac{700 \times 1.00}{1.86 \times 24.92} - 2.0 \times 2.25$ = 10.63 in

s_{max} = 10.63 in > 6.00 in OK

Check Reinforcement Limits

Check Minimum Reinforcement [5.7.3.3.2]

Determine the cracking moment:

$f_r = 0.24\,(f_c')^{0.5}$ = 0.24 x (4.0) $^{0.50}$ = 0.48 ksi

$I_g = (1/12)\,b\,h^3$ = 0.0833 x 12.00 x (6.00) ^{3} = 216.0 in^4

$y_t = (1/2)\,h$ = 0.5000 x 6.00 = 3.00 in

$M_{CR} = \gamma_1\,\gamma_3\,f_r\,I_g / y_t$ = 0.67 x 1.6 x 0.48 x 216.0 / (3.00 x 12.00) = 3.09 k-ft

The capacity of the section must be greater than or equal to the smaller of:

M_{CR} = 3.09 = 3.09 k-ft GOVERNS
$(4/3)\,M_u$ = 1.33 x 3.78 = 5.04 k-ft

The capacity of the reinforcement is:

$M_r = \phi\,A_s\,f_y\,(d_s - a/2)$

$$M_r = 0.90 \times 0.40 \times 60 \times \left(3.75 - \frac{0.40 \times 60}{1.7 \times 4.0 \times 12}\right) \times \left(\frac{1}{12}\right) = 6.22 \text{ k-ft}$$

6.22 k-ft > 3.09 k-ft OK

Use #4 bars at 6.00 in c/c for horizontal lagging reinforcement.

Shrinkage and Temperature Reinforcement [5.10.8]

Not required for components with smaller dimensions less than or equal to 18 in.

PILE SHEAR CAPACITY CHECK

FROM L-PILE ANALYSIS RECEIVED FROM GEOTECH:

SERVICE SHEAR = 21.0 K

STRENGTH I SHEAR = 29.7 K = V_u

SHEAR CAPACITY OF W 27 x 84

REF. AASHTO LRFD-6

$V_u \leq \phi_v V_n$ (6.10.9.1-1) $\phi_v = 1.00$ (6.5.4.2)

$V_n = V_{cr} = CV_p$ (6.10.9.2-1)

$V_p = 0.58 F_{yw} D t_w$ (6.10.9.2-2)

$F_{yw} = 50$ ksi

$D = 25.43$ IN

$t_w = 0.46$ IN

$V_p = 0.58 (50ksi)(25.43IN)(0.46IN)$

$V_p = 339.2$ K

FIND C → (6.10.9.3.2-4)

$D/t_w = 25.43/0.46 = 55.3$

$1.12\sqrt{\frac{Ek}{F_{yw}}}$ WHERE $k = 5.0$ (6.10.9.2)

$E = 29{,}000$

$1.12\sqrt{\frac{29000(5)}{50}} = 60.3$

$D/t_w \leq 1.12\sqrt{\frac{Ek}{F_{yw}}}$ THEREFORE $C = 1.0$ (6.10.9.3.2-4)

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CLIENT ODOT
PROJECT CUY 271
SUBJECT WALL W52
PILE SHEAR CAPACITY

PROJECT NO. 1122-1001-90
SHEET NO. 2 OF 2
COMP. BY ASP DATE 08/27/14
CHECKED BY LNB DATE 9/2/14

PILE SHEAR CAPACITY CHECK (CONT.)

$V_n = CV_p$

$V_n = 1.0\,(339.2k)$

$V_n = 339.2k$

$V_u \le \phi V_n$

$29.7k \le 1.0\,(339.2k)$

$29.7k \le 339.2k$ OK PILE SATISFIES SHEAR CAPACITY